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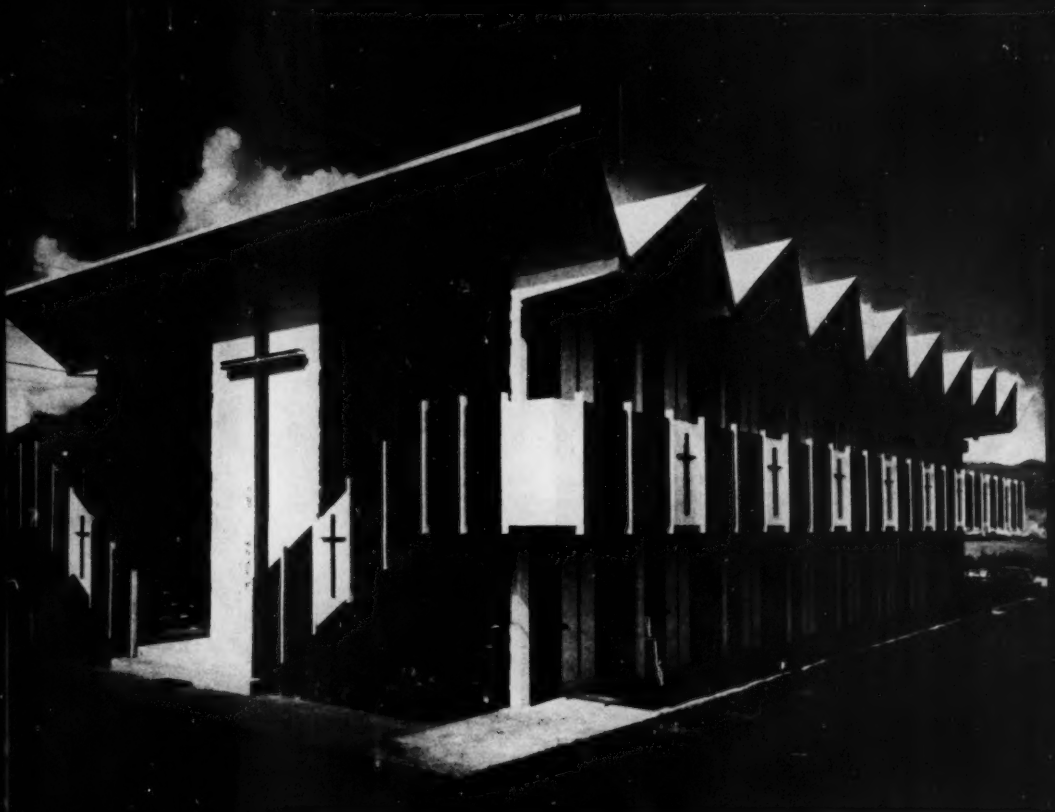

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On the Cover

Kailua Church Education Building, Hawaii. Folded plate roof cast in one continuous operation consists of 36 sections 5 x 22 ft, over-all dimensions 90 x 44 ft. Roof is reinforced with #4 bars at 12 in. on center each way. Remainder of building is precast concrete except for lava rock end walls, wood jalousies, and brightwork. Designer: Wimberly and Cook. Contractor: T. Iida Contractors, Ltd.—Photo by Camera Hawaii, courtesy Permanente Cement Co.

Coming next month in the JOURNAL

RICHARD R. BRADSHAW opens the August issue with "Application of the General Theory of Shells," discussing the construction and behavior of the shell and presenting the general differential equations of a shell with a method of solution for same.

WARREN H. GRAY, JOHN F. MCLAUGHLIN, and JOHN C. ANTRIM report on "Fatigue Properties of Lightweight Aggregate Concrete."

J. O. BRYSON and D. WATSTEIN present research data resulting from a "Comparison of Four Different Methods of Determining Drying Shrinkage of Concrete Masonry Units" to determine their suitability as possible standard test methods.

"Supporting Structure for Retractable Roof of the Pittsburgh Public Auditorium," by EDWARD COHEN and H. REY HELVESTON covers a description of the concrete supporting structure, design conditions, construction of the ring girder and podium, foundation, abutments and anchorages, and the concrete requirements.

R. F. FELDMAN and P. J. SEREDA report on "Characteristics of Sorption and Expansion Isotherms of Reactive Limestone Aggregates."

JOHN N. CERNICA and M. JEAN CHARIGNON discuss research conducted to determine "Plastic Strain in Prestressed Concrete Beams Under Sustained Load."

CHING-SHENG WU, ARPAD A. PAP, and D. H. PLETTA present a report on "Tests of Rigid Frame Bridge Model to Ultimate Load."

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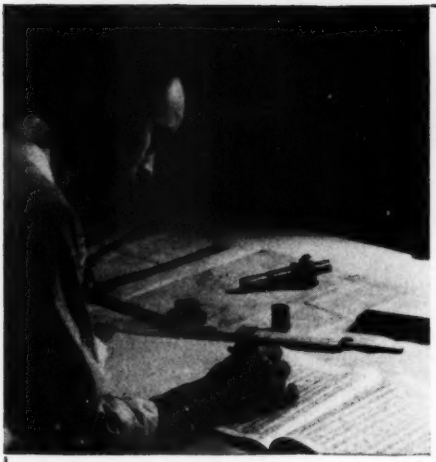

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Tensile Strength and Diagonal Tension Resistance of Structural Lightweight Concrete

By J. A. HANSON

Describes the tests employed and the results obtained in an extension of a previous study of diagonal tension resistance reported by the author. This extension of the original program involves lightweight concrete beams of longer span and lower steel percentages. An important conclusion, that diagonal cracking load should be considered as the ultimate load for non-web-reinforced beams, has been confirmed.

A large number of 6 x 12-in. cylinders from the beam concretes were broken by the "split-cylinder" tension test. Good correlation was established between this indirect tension measurement and the shear resistance of the beams at diagonal cracking. This correlation shows that the diagonal tension resistance of lightweight concretes varies from approximately 60 percent of that of the similar normal weight concrete to nearly 100 percent, depending on the particular lightweight aggregates used.

Proposed ultimate load design recommendations are made for structural lightweight concrete. These are in general accord with the recommendations of the ACI-ASCE Committee 326 on shear and diagonal tension for normal weight concrete. It has been found that diagonal tension strength of the lightweight concretes is affected by the same variables as affect the resistance of normal weight concrete. The difference between the two types of materials is one of magnitude of diagonal tension resistance and not of fundamental difference in behavior.

The proposed design recommendations also provide for the fundamental differences in tensile resistance that exist between the various lightweight aggregates. A combination of compressive strength and split-cylinder tension testing provides a convenient and safe measure of the ultimate diagonal tension resistance to be associated with each of the various aggregates.

Several years ago, Subcommittee 1 of ACI Committee 213, "Properties of Lightweight Aggregates and Lightweight Aggregate Concrete," Truman R. Jones, Jr., Acting Chairman, was assigned the task of collecting material on the properties of structural lightweight concrete and of preparing recommended design procedures for the consideration of other committees of the Institute. This subcommittee is preparing a proposed guide for structural lightweight concrete, but much work remains to be done before the guide can be submitted as a report for approval of the committee as a whole.

One of the more important strength characteristics of structural lightweight concrete is the resistance to diagonal tension of beams without web reinforcement. In recent years, a few structures of both lightweight and normal weight materials have suffered damage due to lack of sufficiently precise design knowledge of this property. ACI-ASCE Committee 326 has completed recom-

mendations for ultimate strength design for shear in normal weight concrete members, and their report urges that ACI Committee 213 complete similar recommendations for structural lightweight concrete. Subcommittee 1 of Committee 213 has gathered sufficient data for formulation of such recommendations.*

This paper presents the information gathered and includes recommended design procedures for diagonal tension in lightweight concrete beams without web reinforcement. These design procedures were developed by the combination of the reported test analyses with Eq. (9) of this paper. Eq. (9) was originally formulated by Committee 326, and the details of its development will be presented in a complete Committee 326 report which is now nearing completion. In addition, the format of the specific design recommendations in the final section of this paper follows that used by the Committee 326.

In the interest of early dissemination of the information, and of obtaining early additional discussion of the subject, particularly as an aid to the ACI Committee 318, Standard Building Code, this paper is published prior to the basic reference report. The paper was prepared under the direction of Subcommittee 1, chiefly from material furnished by the Portland Cement Association. During the 1961 annual ACI convention at St. Louis, Committee 213 unanimously approved sponsorship of this paper and its recommendations for diagonal tension design of lightweight concrete structures.

A complete list of Committee 213 members follows:

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■ FIVE YEARS AGO, as part of a comprehensive study of the characteristics of structural lightweight concrete, the Portland Cement Association tested a series of beams designed to fail in diagonal tension. The results of these tests were reported in 1958.[1] The beams of this earlier study were characterized by high percentages of steel and rather short spans, and the important variable, in addition to type of aggregate, was the compresive strength of the concrete. For several years investigators of diagonal tension have recognized that shear span and steel percentage also have an important bearing on the resistance of beams to shear. In their discussion of the previous paper, Ferguson and Thompson[2] of the University of Texas presented test data on the ultimate shear strength of 12 lightweight beams made with a single ag-

*Report by ACI-ASCE Committee 326 on Shear and Diagonal Tension, Part 2, Chapter 5, unpublished.

ACI member **J. A. Hanson** is senior development engineer, Products and Applications Section, Portland Cement Association, Research and Development Laboratories, Skokie, Ill. He was previously with the U. S. Bureau of Reclamation in Denver, Colo., attached to the Dams Design Division and later to the Structural Research Section of the Bureau laboratories. He is currently a member of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete.

gregate, but with lower steel percentages and longer spans. The indicated shear strengths of the University of Texas beams were lower than those reported in the original paper, and Ferguson and Thompson suggested that the effects of steel percentage and shear span may be more pronounced in lightweight than in normal weight concrete. At a later time, Ferguson furnished the unpublished results[3] of 15 additional beams using the same lightweight aggregate. These later findings corroborated the indicated low diagonal tension resistance of long span, low steel percentage lightweight beams for the particular aggregate used.

With the increasing general demand for inclusion in building codes of provisions for design of lightweight concrete, the original diagonal tension beam program at the PCA laboratories was extended to include the variables of span and percentage of tension steel. The effect of these important variables in the extended program has been studied mainly with concretes having a nominal compressive strength of 4500 psi. Two additional commercially available lightweight aggregates, as well as six of those employed in the original study, have also been included. With these eight lightweight aggregates and the two variables of span and steel percentage, it was not practical to make duplicate beam tests. Thus, close controls on mix proportioning and mix duplication became fundamentally important. In addition to those of lightweight concrete, normal weight beams containing Elgin sand-and-gravel aggregate were tested for comparison of performance. This comparison was made on the basis of equal compressive strengths. For the readers' greater understanding, it is emphasized that this comparison normal weight material is designated as Aggregate 8 throughout this report.

It has been recognized for some time that diagonal tension resistance is fundamentally a function of the tensile strength of the concrete. Previous attempts to establish this relationship by modulus of rupture tests have met with meager success. This is probably due to the sensitivity of the plain concrete beams to shrinkage, temperature, minor specimen defects or other accidental variations. In the present program a measure of the tensile strength of the various concretes was obtained by the "Brazilian" or "split-cylinder" test, in which the cylinder is tested to failure under diametral compression. The cylinders for the splitting test were cast from the beam concretes and cured in the same manner. The concretes used in the 1958 series were repro-

duced to obtain cylinders for correlation with the early beam diagonal tension strengths. These early tests have been reanalyzed and incorporated into this report.

Both of the shear investigations are integral parts of a general study of the properties of lightweight aggregate concrete. Many basic design properties, including compressive and flexural strength, modulus of elasticity, bond, creep, and drying shrinkage of concretes containing these same aggregates, have been reported by Shideler.[4] He has also reported results[5] of a study on the effects of steam curing on creep of concrete. Values of ultimate strength design coefficients for lightweight concretes were given by Hognestad, *et al.*[6] These studies have demonstrated a large variation of properties between different lightweight aggregates, even between those of the same type and manufacture. This indicates the need for individual producers to develop design data through sound investigations of their own products. Fortunately, a simple means of establishing the diagonal tension resistance to be associated with a particular aggregate is suggested by the analysis of this new investigation.

TESTING PROCEDURE

The new investigation was directed toward test specimens with longer spans and more practical amounts of tension steel, and was planned around four types of beams as shown by Fig. 1. All beams were 6 x 12 in. in gross cross section and the spans were either 6½ or 10 ft. Type 1 beams were identical with those of the first investigation. Type 2 beams contained the same tension steel as the Type 1, but were lengthened to provide twice the shear-span length. Type 3 beams retained the shear span of the Type 1 beams but contained only half of the tensile steel area. Finally, the Type 4 beams combined the changes of both the variables, shear span and tensile steel area, into one beam, with twice

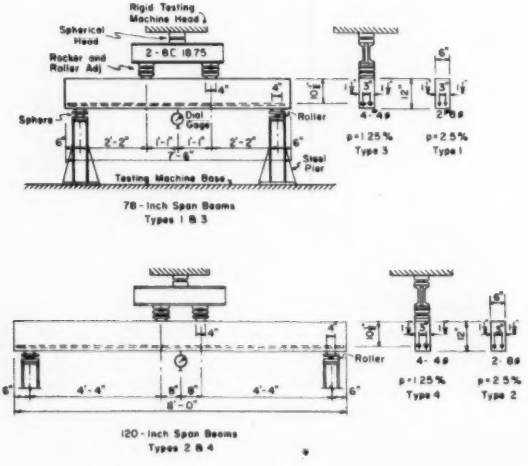


Fig. 1 — Details of beams and testing arrangement

TABLE 1 — PROGRAM OF BEAM TESTS

Beam type	No. of beams	Span, ft	Shear span, a, in.	a/d	p, percent	Nominal f_c', psi
1	9	6.5	26	2.5	2.50	3000
1	10	6.5	26	2.5	2.50	4500
1	3	6.5	26	2.5	5.00	7000
1	2	6.5	26	2.5	5.00	9000
2	9	10.0	52	5.0	2.50	4500
3	9	6.5	26	2.5	1.25	4500
4	4	10.0	52	5.0	1.25	3000
4	11	10.0	52	5.0	1.25	4500
Total	57					

the shear span and half the tensile steel of the Type 1 beams. Web stirrups and compression steel were not used. Table 1 gives the detailed outline of all diagonal tension test beams in both of the programs. There were 36 beams tested during the recent investigation.

Most of the Type 1 beams were cast and tested during the 1958 study. However, three Type 1 beams were added in the recent program using the nominal 4500 psi compressive strength concrete. Two of these contained the two new aggregates, No. 10 and 13, and the third beam was tested to confirm that the high diagonal tension resistance of Aggregate 7 as found in the original study could also be obtained from a later shipment of that aggregate. The four Type 4 beams at $f_c' = 3000$ psi were tested to verify the analysis of the other beams, particularly in regard to the effect of tensile strength on the diagonal tension resistance. Ten beams containing sand-and-gravel aggregate are included in Table 1. These beams were tested to obtain a comparison with the performance of the lightweight beams. Since only one rectangular beam cross section was employed, and also since only symmetrical concentrated loading was employed, the reported performance of the lightweight concrete beams as compared to normal weight beams may then be qualified by the particular dimensions and loading chosen for the test.

Aggregates and concrete mixes

The aggregate identification numbers used here are the same as in the previous reports.[1,4,5] No. 10 and 13 represent the new lightweight aggregates added to the test series. These aggregates, as well as the new shipment of Aggregate 7, were used with their commercial grading. The older aggregates were screened and recombined to the standard grading presented by Shideler,[4] who furnished a complete description of the aggregates. For convenience, an abbreviated description is given below:

Aggregate 2 is an expanded shale produced in a rotary kiln. Material passing No. 4 sieve is obtained by crushing.

Aggregate 3 is an expanded shale produced in a rotary kiln. All particles are rounded and have a smooth shell.

Aggregate 4 is an expanded clay produced in a rotary kiln. Most of the material is crushed but is not harshly angular.

Aggregate 5 is an expanded slate produced in a rotary kiln. The coarser particles are somewhat angular and porous. Material passing No. 4 sieve is obtained by crushing.

Aggregate 6 is an expanded blast furnace slag produced by spraying a controlled amount of water on a thin layer of molten slag. All particles are angular and porous and most of the finer sizes are obtained by crushing.

Aggregate 7 is produced by burning on a sintering grate a carbonaceous shale from anthracite coal processing. All sizes are obtained by crushing and particles are sharp and porous.

Aggregate 9 is an experimental expanded shale made in a rotary kiln. Particles are rounded and have a smooth shell.

Aggregate 10 is an expanded shale produced in a rotary kiln. The particles are generally rounded and sealed.

Aggregate 13 is an expanded shale produced in a rotary kiln. All particles are rounded and have a smooth shell.

Aggregate 8 is *normal weight* Elgin sand and gravel. The gravel is well rounded.

The concrete placing and proportioning procedures for the new beams were also similar to those for the plain concrete tests and for the original shear study. Details may be found in Reference 4. For comparison purposes the final mix quantities, plastic unit weights, and slumps of the new concrete mixes are given in Table 2.

TABLE 2 — MIX PROPORTIONS OF CONCRETES FOR DIAGONAL TENSION BEAMS

Aggregate No.	Fine aggregate, percent by volume	Quantities per cu yd of concrete					Percent air (Rollometer)	Plastic unit weight of concrete, lb per cu ft	Slump, in.
		Water, lb	Cement		Aggregate				
			Sacks	lb	Fine, lb	Coarse, lb			
3000 psi Series A									
2	50	442	6.06	570	785	638	7.0	90.2	2⅛
3X	45	332	4.41	415	980	754	7.0	91.9	2
3	45	346	5.41	509	991	766	6.4	96.7	2⅛
4	55	498	4.22	397	1048	750	5.9	99.7	2⅛
5	55	423	5.62	528	1098	606	7.2	98.3	1½
6	55	398	6.53	614	1187	650	8.2	105.5	1⅛
7	55	424	5.98	562	1136	630	7.0	101.9	1⅛
10	39	350	4.04	380	764	918	6.2	89.3	2
8	48	196*	3.63	341	1588*	1720*	5.9	142.4	3¼
4500 psi Series B									
2	45	450	8.82	829	653	650	5.9	95.6	2¼
3	40	360	8.50	799	751	713	5.3	97.2	2⅛
4	50	472	5.72	538	927	811	5.9	101.8	2⅛
5	50	418	8.02	754	969	655	6.0	103.0	2⅛
6	50	402	8.96	842	1045	698	6.1	110.6	2
7	50	443	8.82	829	961	649	5.9	106.7	2⅜
10	36	346	5.71	537	678	926	6.0	92.1	2⅜
13	55	372	8.53	802	855	518	5.9	94.3	2⅛
8	39	205*	5.31	499	1241*	1842*	5.9	143.9	2⅝
7000 psi Series C									
4	35	454	6.79	638	705	1100	2.0	107.3	½
8	25	224*	5.85	550	843*	2528*	1.1	153.5	⅝
9000 psi Series D									
4	35	475	10.06	946	622	974	2.0	111.7	¾
8	25	276*	9.93	933	727*	2182*	1.1	152.5	⅞

*Net water—Saturated surface dry aggregate.

TABLE 3 — AVERAGE YIELD STRENGTH AND MODULUS OF ELASTICITY

Grade of steel	Bar size	Yield point		Modulus of elasticity	
		No. of samples	f_y, psi	No. of samples	E, psi
Intermediate	#8	5	47,300	4	28,200,000
	#4	31	48,450	5	27,750,000
High strength	#7*	1	93,500	—	—
	#4	10	88,580	—	—

*Two pairs of one #7 and one #4 bundled reinforcement were used as equivalent to two #8 bars in Beam 8B2.

The mix quantities shown for the lightweight concretes have been computed on the cement content, dry aggregate basis in accordance with ACI 613A-59.* Most of the Series A and all of the Series C and D mixes were cast into 6 x 12-in. cylinders only. The 1958 report should be consulted for mix details of the original beam series. The Series A mix designated 3X was cast to produce split cylinders for Aggregate 3 at a lower strength level than was obtained with the beam mix labeled Aggregate 3.

Reinforcing steel

Preliminary estimates prior to the performance of the tests indicated that the intermediate grade reinforcing steel used in the lightweight beams would have a sufficiently high yield point to prevent flexural failure prior to diagonal tension cracking. These same estimates indicated that high strength steel should be employed in the sand-and-gravel beams. This same precaution should have been observed in the Type 4 beams containing two of the lightweight aggregates, these beams indicating flexural rather than diagonal tension failure. The deformations of all bars were in accordance with ASTM A 305. Samples of the reinforcement were tested at intervals throughout the duration of the program. Average yield strengths and moduli of elasticity are given in Table 3.

Fabrication and curing of specimens

All beams were cast in plywood forms. The reinforcing bars were supported from the bottom form by metal chairs at a height to provide an effective depth of 10½ in. for all beams. The four #4 bars used in the Types 3 and 4 beams were bundled* in pairs to provide clearance for the larger size aggregate during concreting.

The concrete was batched by weight and mixed in a 1¾ cu ft pan-type mixer. In accordance with ACI 613A-59 the aggregate and two-thirds of the mixing water were first introduced into the mixer and mixed for 2 min; then the cement, air-entraining agent and the remaining water were added and mixing was continued for an additional 3 min. A blend of four commercial Type I cements was used.

The concrete was placed in the beam form in four 3-in. lifts. Immediately after casting each beam, additional 1½-cu ft batches of concrete were mixed for fabrication of 6 x 12-in. cylinders for compressive strength and tension splitting. The concrete in all beams and cylinders was consolidated by internal

*ACI Committee 613, Subcommittee on Proportioning Lightweight Aggregate Concrete, "Recommended Practice for Selecting Proportions for Structural Lightweight Concrete (ACI-613A-59)," American Concrete Institute, Detroit, 1959, 10 pp.

vibration, with the beam concrete being vibrated after placement of each second lift. The steel cylinder molds were covered with ground steel plates for the first 24 hr after casting and the beams were covered with a plastic film. At the end of 1 day, the forms and molds were stripped and the specimens were placed in a moist room (73 F) until 7 days. At this time, all beams, all cylinders for compression testing, and one-half the cylinders for tension splitting were stored in laboratory space controlled at 73 F, 50 percent relative humidity, until tested. The other half of the tension splitting cylinders remained in the moist room until tested in the wet condition. All tests were performed at 28 days.

Testing procedure

The beams were tested in a 1,000,000 lb hydraulic compression machine. Fig. 1 shows the details of the beam fabrication and the testing arrangement. The 4-in. wide reaction and load bearing plates were seated and leveled in thickened polyester resin to assure full bearing. A dial gage located at center-span indicated maximum deflection of the simply supported beams. The light-weight concrete beams were loaded in increments of 1000 lb and the normal weight beams in 2000-lb increments. At each increment the load was maintained for a period sufficient to record deflections and mark the progression of cracks. About 45 min were required for each beam test.

A 400,000 lb universal hydraulic testing machine was employed for the split-cylinder test. This machine was fitted with a 12 in. wide rectangular head on a spherical bearing. The testing machine center line was carefully marked on each side of the 12-in. dimension of this head. The concrete cylinder was placed on the testing machine platen on top of a plywood strip, as shown by the upper view of Fig. 2. The machine crosshead was then adjusted close to the concrete cylinder. After determining that the ends of the cylinder coincided with the sides of the bearing block, the cylinder was adjusted sideways until the machine center-line was in alignment with the vertical diameter. No premarking of the vertical diameter was made, but the alignment was judged by eye when the machine center line passed through the point of horizontal tangency to the cylinder. Another plywood pad was then inserted between the cylinder and the testing machine head. These plywood pads were strips ⅛ in. thick, 2 in. wide, and a full 12 in. long, cut from sheets to which a kraft paper was bonded on both sides.

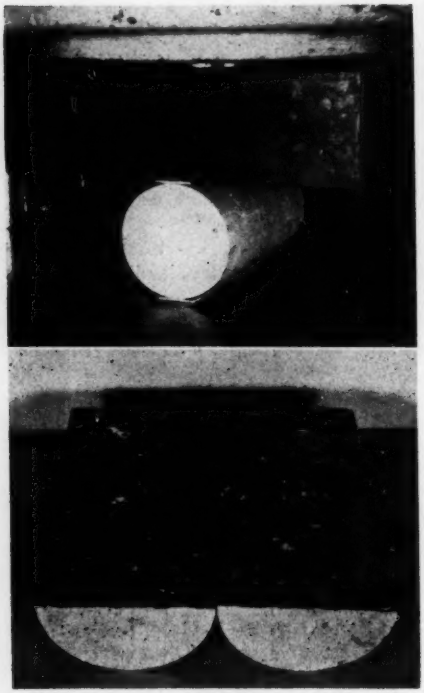


Fig. 2—Cylinder splitting test in progress (top); lightweight concrete cylinder after tensile failure (bottom)

One of the European authorities, Thaulow,[9] has recommended that the tension cylinder be loaded at approximately one-fourth the rate of a corres-

ponding compression cylinder. Consequently, a loading rate of 14,000 lb per min was established for these tests. This rate was applied immediately after contact with the head was established, and the loading was continued until needle drop and vertical splitting occurred simultaneously. It is of interest to note that examination of the pads showed that approximately a ½ in. length of arc was in contact with the cylinder.

TEST RESULTS AND DISCUSSION

Split-cylinder tension tests

The Brazilian or split-cylinder test for tensile strength of concrete may have been first introduced by Lobo B. Carneiro and Barcellos.[10] This particular test has found considerable favor in other parts of the world, but is not widely used in the United States. However, a few investigations of the adequacy of this simple measure of tensile strength are continuing in this country, one of which was discussed in the Research Session of the 1960 ACI annual convention. Thaulow presents a good bibliography of foreign theoretical and experimental studies. Investigators have studied such test variables as water-cement ratio, size of aggregate, age of concrete, size of specimen, rate of loading, and size and thickness of loading pads. All of these variables will affect the test results to a greater or smaller degree.

Rüsch and Vigerust[11] have presented perhaps the most effective discussion of the reliability and usefulness of the tensile splitting test. In addition to the practical advantage of ease of testing, they discuss several important advantages of this method over the flexure test as a measure of tension. They point out the sensitivity of the outer fibers of the flexure specimen, which sustain the maximum stress, to incidental moisture changes, while the maximum tension stresses in the split-cylinder test are applied to the interior of the cylinder. A similar effect results from minor local defects such as large pieces of aggregate near the surface. They also discuss the increased mathematical probability that weak points with low tensile resistance will fall within an area of maximum stress in the flexure test, while the split-cylinder test is virtually independent of the dimensions of the test specimen. These two authorities conclude that the split cylinder is easier to test; that the dispersion of the test data will be less for the tensile splitting test; and that the indicated tensile strength obtained from the cylinder splitting test will not deviate far from the true tensile strength of the material.

There is one major conclusion that may be drawn from all the various investigations reported on this indirect measurement of tensile strength. This is that the split cylinder can furnish a reliable measure of the *relative* tensile strength of concrete, provided that several samples are employed.

The theory of elasticity indicates that a uniform tensile stress at right angles to the direction of load application must exist over a substantial

part of the interior of a cylinder subjected to concentrated loads at the opposite ends of a diameter. This stress is given by the formula

$$f'_{sp} = \frac{2P}{\pi DL} \quad \text{(1)}$$

where f'_{sp} is the uniform tensile stress, P is the magnitude of load, and D and L are the cylinder diameter and length, respectively.

The tensile strength determined in this manner from measured values of P may be subject to influence from test conditions such as cylinder size, loading rate, and pad dimensions. The stress condition in the cylinder is obviously biaxial. The theory shows that the compressive stress at the center of the cylinder, and parallel to the loaded diameter, is equal to three times the tensile stress. The average compressive stress over the loaded diameter may equal five or six times the uniform tensile stress, depending on the conditions of load distribution and plasticity under the concentrated loads.

For the above reasons, the stress computed by Eq. (1) should probably not be considered as the true tensile strength of concrete. However, a study of the various investigations show that the split-cylinder test is a good measure of the relative tensile strength and that this test will reliably reflect such variables as compressive strength, water-cement ratio, age of concrete, and type of curing. Some advantages of the split-cylinder test over the modulus of rupture test have already been discussed but perhaps the most important advantages of the split cylinder are the following two considerations. First, the tensile stress distribution must approach uniformity over a large diametral area of the cylinder, and thus stress concentrations from specimen defects or other causes will be reduced by plasticity. Secondly, the test procedure is extremely simple and affords the opportunity to economically test a large number of specimens, thus offsetting the rather large variation of results that must always be expected in tensile investigations.

In connection with the extended diagonal tension program, 225 lightweight concrete cylinders and 57 normal weight concrete cylinders were broken by the splitting test. Photographs of this test, in progress and after failure, are shown in Fig. 2. Approximately one-half of the specimens of each type of concrete were given 7 days moist curing followed by 21 days drying at 73 F and 50 percent relative humidity. The other half of the specimens were continuously moist cured at 73 F. The break of the cylinder was always an approximate plane located on or near to the loaded diameter. In all lightweight concrete cylinders, regardless of compressive strength, this failure plane passed directly through nearly all pieces of aggregate. In the lower strength sand-and-gravel specimens (Series A and B), only 10 to 20 percent of the aggregate pieces were broken. In the extremely high strength sand-and-gravel cylinders (Series C and D) practically all aggregate was ruptured. It

would appear then that tensile strength of lightweight concrete is largely determined by the tensile strength of the particular aggregate, while the tensile resistance for the usual ranges of strength of normal weight concrete is determined by bond of the paste to the aggregate. Only after the compressive strength reaches approximately 7000 psi or above, will the tensile strength of this particular sand-and-gravel concrete be primarily affected by tensile strength of the aggregate.

The measured split-cylinder tensile strengths of all the concretes are presented in Table 4. These results include cylinders of approximately the same compressive strength as the concretes in the original shear beam series[1] as well as from the present study. The table also presents the number of cylinders tested and the coefficients of variation of the measurements for each concrete. The coefficient of variation has been computed by an approximate method[12] for a small number of samples (10 or less). Examination of the table shows, with only a single excep-

TABLE 4 — TENSILE STRENGTH OF LIGHTWEIGHT CONCRETES AS MEASURED BY THE SPLIT-CYLINDER TEST

Aggregate No. and strength series	f_c', psi	Dry concrete*							Moist concrete*			
		No. of specimens	Avg f'_{sp}, psi	Coefficient of variation, percent	f'_{sp}/f_c', percent	a_1†	Calculated f'_{sp}	$f'_{sp\ test}/f'_{sp\ calc}$	No. of specimens	Avg f'_{sp}, psi	Coefficient of variation, percent	f'_{sp}/f_c', percent
2 A	3210	6	265	8.0	8.3	4.28	243	1.09	6	365	6.0	11.4
B	4840	6	271	6.3	5.6		298	0.91	6	434	9.9	9.0
3 AX	3000	6	269	8.9	9.0	4.84	265	1.02	6	357	10.4	11.9
A	3850	5	299	6.9	7.8		300	1.00	4	428	4.8	11.1
B	4320	5	314	6.9	7.3		318	0.98	4	452	7.5	10.5
4 A	3100	5	294	13.5	9.5	4.80	267	1.10	5	359	6.7	11.6
B	5090	6	307	3.5	6.0		342	0.90	5	394	21.2	7.7
C	7100	6	348	13.8	4.9		405	0.86	6	593	8.1	8.4
D	8100	6	372	11.8	4.6		432	0.86	6	598	10.2	7.4
5 A	3260	6	323	10.2	9.9	5.28	301	1.07	5	361	4.3	11.1
B	4940	6	345	9.4	7.0		371	0.93	6	471	11.8	9.5
6 A	3320	6	302	2.7	9.1	4.96	286	1.06	6	362	10.9	10.9
B	4890	8	327	6.5	6.7		346	0.94	7	471	11.3	9.6
7 A	3280	6	348	10.6	10.6	5.96	342	1.02	6	351	8.4	10.7
B	4840	8	408	7.9	8.4		415	0.98	9	479	8.6	9.9
10 A	3270	6	273	6.2	8.4	4.55	260	1.05	5	394	5.8	12.0
B	4700	9	297	5.7	6.3		312	0.95	10	472	10.2	10.0
13 B	5040	8	276	10.8	5.5	3.89	276	—	9	439	12.7	8.7
8 A‡	3030	7	380	3.2	12.5	6.88	378	1.00	6	324	6.5	10.7
B	4380	10	454	7.5	10.4		455	1.00	10	415	9.4	9.5
C	7300	6	687	9.1	9.4		588	1.17	6	618	5.0	8.5
D	8600	6	663	6.5	7.7		638	1.04	6	672	10.6	7.8

*Dry concrete indicates 7 days moist curing followed by 21 days drying at 73 F, 50 percent relative humidity. Moist concrete indicates 28 days moist curing.

† $a_1 = f'_{sp}/\sqrt{f_c'}$

‡ Aggregate 8 is Elgin sand and gravel.

tion, that the coefficient of variation of the different concretes varies from 3 to 14 percent, with many values between 6 and 10 percent. These values may appear to be rather large, but it must be remembered that tension tests always have a large variation.

Relation of tensile strength to compressive strength

Table 4 also shows the split-cylinder tensile strength as a percentage of the compressive strength. The program did not include 28 day moist-cured compression tests, so the assumption was made that the 28 day compressive strengths of the dry and moist cured concretes were equal. This assumption is well justified from an examination of Shideler's[4] data on similar concretes. For the dry concretes (7 days moist curing, 21 days drying at 73 F, and 50 percent relative humidity) the tensile strengths of the lightweight concretes range from approximately 9.5 percent of 3000 psi compressive strengths down to some 7 percent at the 4500-psi level. These percentages approach 4.5 percent of the extremely high strengths of the Aggregate 4 concrete. Corresponding approximate percentages for the sand-and-gravel concrete are 12.5, 10.5, and 7.5. The ratio of the moist split-cylinder tensile strengths to the compressive strengths for lightweight concrete averages about 2 to 3 percentage units higher than the corresponding ratio with dry tensile strengths. This comparison is reversed for the normal weight concretes where the "wet" percentages are one to two units lower than the dry. Certainly the data indicate that tensile strength is not a constant proportional part of the compressive strength, but that the proportionality ratio decreases as compressive strength increases. This trend agrees with the findings of Rüsch and Vigerust[11] who tested concrete that had dried for 3 weeks. Similar verification is furnished by Gruenwald[13] for continuously moist cured concrete.

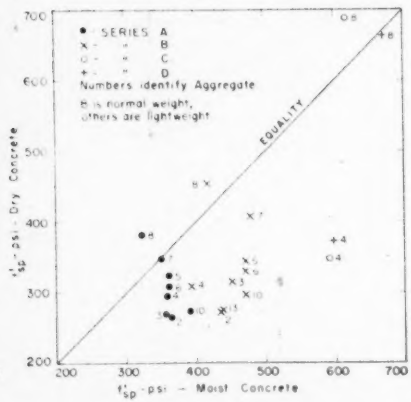


Fig. 3—Relationship of moist and dry split-cylinder tensile strength

Since tensile strength then cannot be considered as a linear proportion of compressive strength, many investigators have related tensile resistance to the square root of the compressive strength, such as:

$$f'_{sp} = \alpha_1 \sqrt{f_c'} \quad \text{.....(2)}$$

Table 4 presents values of α_1 for the dry concretes containing the various aggregates together with calculated values of f'_{sp} for each compressive strength. The ratio of measured f'_{sp} to calculated f'_{sp} reveals that the average α_1 computes a tensile strength that is too

low for the lower compressive strength lightweight concretes and too high for the higher strengths. However, this variation averages only ± 6 percent. Eq. (2) is then not entirely unsatisfactory for lightweight concretes provided that the value of α_1 is considered as a function of the aggregate. The relation appears quite satisfactory for the normal weight concretes of usual compressive strengths.

A most important observation from Table 4 is that the relative level of tensile resistance of each concrete is a definite characteristic of each particular aggregate, when the dry concretes are compared on an equal compressive strength basis.

Effect of drying on tensile strength

The dry split-cylinder tensile strengths of Table 4 are plotted in Fig. 3 as a function of the corresponding tensile strengths for continuous moist curing. It will be noted that the moist tensile strengths of each of the four series fall in a rather narrow band; the dry tensile strengths, however, vary widely within each series. Also, the dry tensile strengths of the concretes containing each aggregate fall in the same order of progression. This again reflects that the magnitude of dry tensile strength is a characteristic peculiar to the concrete containing each particular aggregate.

Examination of Fig. 3 also shows that drying of the lightweight concrete decreases the tensile strength by amounts up to 40 percent depending on the aggregate and the compressive strength. It might be surmised that this effect is due to tensile stresses imposed by drying shrinkage, but attempts to correlate this with shrinkage measurements on similar concretes were not successful.

In opposition to the effect on the lightweight concretes, drying of the sand-and-gravel concretes (Aggregate 8) generally increased the split-cylinder tensile strength by amounts up to 15 percent. No explanation is offered for this, unless it might be that stresses induced by drying act to increase the bond of paste to aggregate.

Split-cylinder strength and modulus of rupture

The relationship of tensile strength data obtained by plain concrete beams and by cylinder splitting tests is shown in Fig. 4. The modulus of rupture data were gathered by Shideler[4] on similar concretes. A generally linear relationship between the two test methods is suggested by the data, but the scatter of the points is large, probably because of the sensitivity of the modulus of rupture of partially dried beams to extraneous test conditions. Evidence of this erratic test condition is found in Fig. 4 where it is shown that the tensile splitting values of the *dry* lightweight concretes are larger than the corresponding measured moduli of rupture. Theoretical considerations have led most investigators to regard the modulus of rupture as somewhat larger in magnitude than either the simple or split-cylinder tensile strength. The anomaly can then

TABLE 5 — RESULTS OF DIAGONAL TENSION BEAM TESTS

Beam No.	f_c', psi	f'_{sp}, psi	P_{c_1}, kips	P_{c_2}, kips	P_u, kips	v_c, psi
Type 4 beams, $p = 1.25$ percent — $M_{max}/Vd = 5.0$						
3A4	3850	299	12.00	—	12.00	95
4A4	3100	294	11.70	—	11.70	93
10A4	3270	273	12.90	—	12.90	102
8A4	3030	380	15.20	—	15.20	121
2B4	4940	271	10.90	11.50	11.50	87
3B4	4400	314	11.55	—	11.55	92
4B4	5120	307	12.00	—	13.15	95
5B4	5010	345	—	—	13.80*	—
6B4	4850	327	13.00	—	13.00	103
7B4	4860	408	—	—	15.10*	—
10B4	4850	297	12.40	—	12.40	98
13B4	5150	276	13.20	—	13.20	105
8B4	4490	454	19.25	—	19.25	153
10BW4	4530	—	11.60	—	11.60	92
8BW4	4300	—	18.00	—	18.00	143
Type 2 beams, $p = 2.5$ percent — $M_{max}/Vd = 5.0$						
2B2	4880	271	15.00	—	15.00	119
3B2	4540	314	16.00	—	16.00	127
4B2	5100	307	16.00	—	16.00	127
5B2	4930	345	19.80	—	19.80	157
6B2	5000	327	17.00	—	17.00	135
7B2	4960	408	22.00	—	22.00	175
10B2	4430	297	16.65	—	16.65	132
13B2	5060	276	15.00	17.00	17.25	119
8B2	4470	454	23.56	—	23.56	187
Type 3 beams, $p = 1.25$ percent — $M_{max}/Vd = 2.5$						
2B3	4780	271	14.90	14.90	26.95	118
3B3	4170	314	16.00	16.94	30.02*	127
4B3	5020	307	13.90	17.00	29.70*	110
5B3	4900	345	20.00	20.00	29.70*	159
6B3	4820	327	16.80	16.80	19.00	133
7B3	4870	408	21.85	24.10	25.00	173
10B3	5020	297	17.00	18.35	31.40*	135
13B3	5020	276	16.40	17.00	26.00	130
8B3	4370	454	19.00	20.70	20.70	151

Beam No.	f_c', psi	f'_{sp}, psi	P_{c_1}, kips	P_{c_2}, kips	P_u, kips	v_c, psi
Type 1 beams, $p = 2.5$ percent — $M_{max}/Vd = 2.5$						
2A1	3680	267	19.20	19.20	37.00	152
3A1	3310	280	19.80	—	19.80	157
4A1	2980	293	18.45	20.81	24.71	146
5A1	3490	327	21.71	—	21.71	172
6A1	3670	313	19.75	20.00	20.05	157
7A1X	3210	345	25.46	—	25.46	202
7A1	4240	386	27.72	27.90	39.75	220
8A1X	3700	419	24.06	28.05	36.16	191
8A1	4020	436	25.95	—	25.95	206
2B1	5350	273	17.97	21.18	43.22	143
3B1	4090	300	19.82	20.25	22.00	157
4B1	4890	305	21.56	22.00	37.69	171
5B1	4790	344	21.90	—	21.90	174
6B1	4870	322	19.47	21.56	31.55	155
7B1X	4680	408	28.50	29.65	35.50	226
7B1	5200	417	28.04	29.80	39.40	223
10B1	4860	297	20.00	23.00	43.00	159
13B1	4940	276	18.00	21.10	39.90	143
8B1	5380	502	28.50	33.70	40.64	226
Type 1 beams, $p = 5.0$ percent — $M_{max}/Vd = 2.5$						
9C1	6910	340	26.00	28.40	50.80	206
4C1	7000	348	24.40	28.00	49.10	194
8C1	8410	663	37.74	40.00	57.24	300
4D1	8160	372	28.25	28.25	53.50	224
8D1	10,680	625	39.30	40.00	74.40	312

f_c' = compressive strength of 6 x 12-in. cylinders

f'_{sp} = split-cylinder tensile strength

P_{c_1} = load causing diagonal crack at one end of beam

P_{c_2} = load causing diagonal crack at other end of beam

P_u = ultimate load capacity of beam

$v_c = V/bd$ = unit shear based on first diagonal cracking load

*Denotes failure by yielding of tensile reinforcement

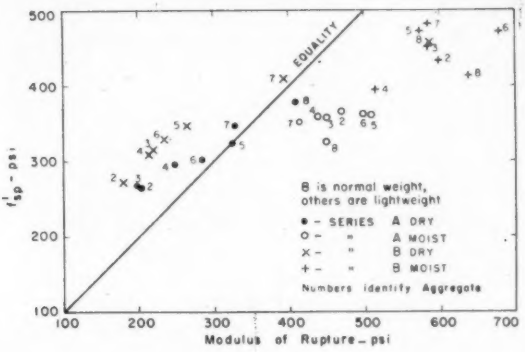


Fig. 4—Relationship of modulus of rupture and split-cylinder tensile strength

be interpreted as further evidence of the deleterious effect of drying on the extreme fibers of the plain concrete beam.

Diagonal tension beam tests

Table 5 presents the test data on diagonal cracking, ultimate load, and unit shear of the diagonal tension test beams. The table includes the 21 beams that were tested in the original program[1] with the unit shears of these beams recomputed according to the following definition:

$$v_c = \frac{V}{bd} \qquad (3)$$

where v_c is the nominal unit shear at diagonal tension cracking, V is the total external shear, and b and d are the width and effective depth of the beam.

This definition, which eliminates the factor j from the denominator, has been proposed by Bower and Viest[14] and Committee 326. The identification numbers of the beams have been coded as follows: the first number is the aggregate identification number described previously (8 being the normal weight aggregate), the following letter shows the nominal compressive strength of the concrete as given by Table 2, and the last number is the beam type defined in Fig. 1. Thus the Beam 4B3 indicates a beam containing a concrete with a rotary kiln expanded shale aggregate, of 4500 psi nominal compressive strength, and a beam containing 1.25 percent steel area over a shear span of 26 in. It will be noted that all of the 21 original beams are Type 1, thus this number has been merely added to the original identification number. The only new Type 1 beams cast in the present investigation were 7B1X, 10B1, and 13B1. Any numbers labeled with an "X" indicates a recasting and retesting of a beam to verify previous results or to adjust compressive strength. The two beams labeled 10BW4 and 8BW4 were tested in the moist condition at 28 days.

The compressive strengths of Table 5 are the average strengths measured by 6 × 12-in. cylinders that accompanied each particular beam.

Fig. 5 — Typical cracking patterns of longer span beams

The dry split-cylinder tensile strengths, f'_{sp}, are the average of 5 to 10 tests of cylinders that accompanied several beams. The tensile strength values for all new beams are identical with Table 4, while, for the early beams, the values of Table 4 have been adjusted to compensate for changes in compressive strength. The values of P_{c1}, P_{c2}, and P_u are the total loads on each of the beams at first diagonal cracking, second diagonal cracking (if such occurred), and at ultimate failure of the beam. These loads were indicated in every case by a drop of the testing machine load-indicator needle.

Ultimate strength of test beams

The flexural cracking of all beams developed in a normal manner. These vertical cracks first appeared in the zone of pure moment, and as the load was increased to higher levels they spread to the areas outside the symmetrically applied loads. In many beams, the flexural cracks outside the pure moment zone inclined toward the loading points just prior to formation of the diagonal crack. The diagonal cracks formed suddenly and resulted in either complete failure of the beam or in sizable loss of applied load. Typical cracking patterns of both long and short span beams are shown in Fig. 5 and 6.

Measured ultimate load capacities of all beams are listed in Table 5. Comparison of the ultimate load, P_u, with P_{c1}, the load at first diagonal cracking, indicates that 19 out of the 24 longer span beams (Types 2 and 4) failed completely at diagonal tension cracking. All of the Type 3 beams ($p = 1.25$ percent) achieved redistribution of forces and absorbed loads of variable amounts above that causing diagonal cracking. As noted in the earlier paper,[1] the diagonal cracking load was ultimate for five of the Type 1 beams ($p = 2.5$ percent). This somewhat erratic ultimate behavior of the beams confirms the conclusion of the earlier paper and of ACI-ASCE Committee 326 that only the load at diagonal cracking should be considered as the ultimate load in the design of non-web-reinforced beams.

Two of the Type 4 beams, those containing Aggregates 5 and 7, failed by yielding of the longitudinal reinforcement prior to formation of diagonal cracks. The high resistance to diagonal tension of concretes containing these aggregates will be discussed later in this report. Four of

Fig. 6 — Typical cracking patterns of short span beams, $p = 1.25$ percent

the Type 3 beams also failed ultimately by yielding of the steel *after* formation of the diagonal cracks. These six beams are indicated by an asterisk accompanying the ultimate load value. A comparison of the measured ultimate flexural moments of these beams with the moments computed prior to testing showed that measured moments vary from approximately 0 to 12 percent higher than the computed, indicating the good reliability of the ultimate design coefficients used in the computation.

Location of diagonal cracks

Both Bower and Viest[14] and the Committee 326 have assumed that the section of potential initial diagonal cracking is located at a distance equal to the beam effective depth d, outside of the point of load application. For short shear spans, this distance is limited to no greater than one-half the shear span. Such an assumption is necessary to the complete functioning of their proposed design formulas. Fig. 7 indicates this assumed location relative to the diagonal crack location for the various beams of nominal 4500 psi compressive strength. The crack locations were traced from photographs after completion of each test. It would appear that this design assumption might agree with an over-all average crack location with a rather large variation in either direction for individual beams. This situation is not surprising since past experience has shown that diagonal crack location is highly variable and in many ways a matter of chance.

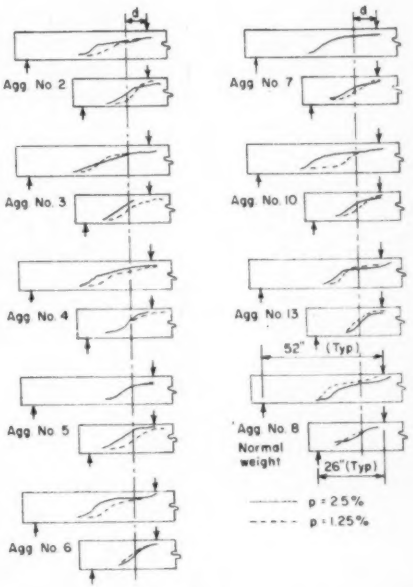


Fig. 7—Location of diagonal cracks in Series B beams

Nominal shear strength

The present ACI Building Code (ACI 318-56) directs that unit shear stresses, v, shall be computed by the formula

$$v = \frac{V}{b\,j\,d} \qquad (4)$$

where V is the total external shear at any section and j is the ratio of the internal resisting-moment arm to the effective depth.

The Code also provides that the allowable working stress in shear in non-web-reinforced beams shall be $0.03\,f_c'$ with an upper limit of 90 psi. Fig. 8 presents a comparison of the unit shears of the test beams computed by Eq. (4) with the code allowable stresses. For convenience, the factor j has been assumed as 7/8. This value is somewhat high for most of the lightweight beams, particularly those with larger steel percentages, and thus the plotted values of unit shear of these beams are somewhat low. These unit shears represent *ultimate* values (at diagonal cracking) and the figure shows that many beams have safety factors with respect to the Code allowable approaching 1.0. Even the normal weight beam (Aggregate 8) with the lowest steel area has a safety factor of only 1.5. All of the Type 4 beams and most of the beams in the Types 2 and 3 categories have measured nominal shears of less than twice the Code allowable. The need for more reliable code provisions for non-web-reinforced beams is obvious. This has been recognized for some time, and Committee 326 is recommending ultimate design procedures for

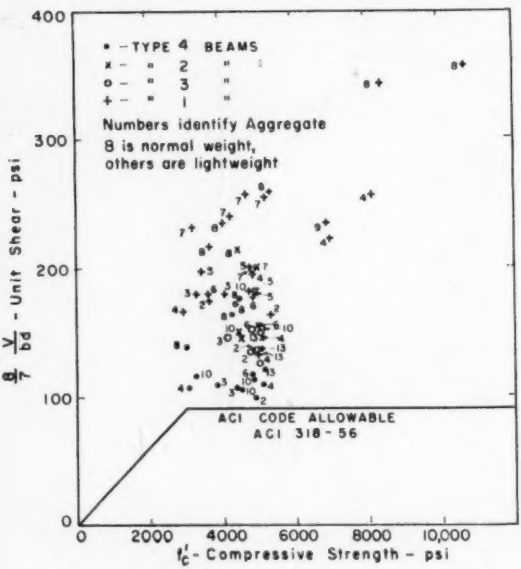


Fig. 8—Measured unit shear as compared with code allowable stresses

diagonal tension in normal weight concrete. A similar approach for lightweight concrete will be discussed in the final section of this paper.

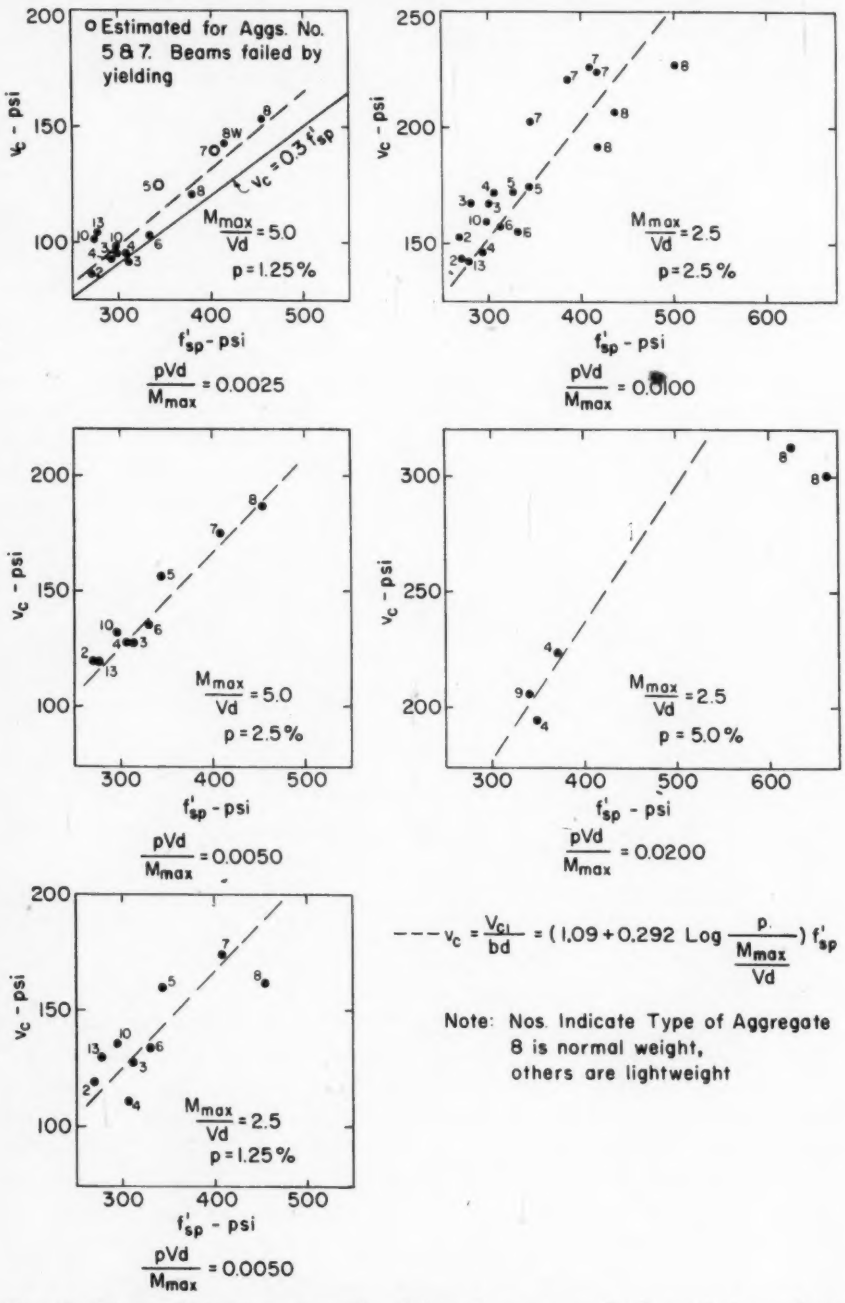
Correlation of unit shear with tensile splitting strength

The Committee 326 report* presents an equation that relates shearing stress to the diagonal tensile strength of concrete. This equation was derived by Morrow and Viest in working toward a rational relationship for the diagonal cracking load starting from the formula for principle stress at a point. The derivation is also based on the theoretical linear stress distribution for reinforced concrete. The resulting equation is quite complex and simplification is obtained by rationalizing that the resistance of concrete to principal tensile stress is comparable to the tensile strength of the concrete. The tensile strength of the concrete in turn is assumed to be directly proportional to the square root of the compressive strength. In the present investigation, the relative tensile strength has been measured by the cylinder splitting test. The detailed results of these tests were presented in Table 4 and the tensile resistance, f'_{sp}, to be associated with each particular beam was given by Table 5.

Detailed examination of Table 5 in regard to both f'_{sp} and v_c, the unit shear computed by Eq. (3), reveals that these quantities fall into a typical sequence with respect to the various aggregates. Further comparison of the two values indicates that the relationship between v_c and f'_{sp} is linear for each of the beam types. This direct relationship and typical sequence of tensile resistance is illustrated in Fig. 9. The dispersion of data shown in this figure appears quite small, particularly when compared to diagrams such as Fig. 8, where unit shear is plotted against compressive strength. This reduction in the spread of diagonal tension data definitely indicates the utility of the split-cylinder tension test as related to the basic cause of diagonal tension failure, i.e., tensile stress that exceeds the tensile strength of the concrete.

The five diagrams of Fig. 9 show the relationship between v_c and f'_{sp} for each of the five values of $pd/(M_{max}/V)$ available in the test program. The term, M_{max}/V, is identical to the shear span length, a, for simple beams with a single concentrated load or with two symmetrical concentrated loads. Each plotted point is labeled with the particular aggregate identification number involved. The typical sequence of points in each diagram indicates the characteristic value of tensile (and thus shear) strength associated with each aggregate. Particular attention is directed to the relative position of the points for Aggregates 6, 5, and 7 which fall in the intermediate to high range, respectively. The sand-and-gravel concrete (Aggregate 8) shows the highest resistance but is often approached in magnitude by Aggregate 7. Pertinent evidence of the high tensile resistance of some lightweight aggregates is indicated by the

*Unpublished.



$$--- v_c = \frac{V_{ci}}{bd} = (1.09 + 0.292 \text{ Log } \frac{p}{\frac{M_{max}}{Vd}}) f'_{sp}$$

Note: Nos. Indicate Type of Aggregate
8 is normal weight,
others are lightweight

Fig. 9—Diagonal tension strength of beams versus split 6 x 12 in. cylinder tensile strength

yielding failure of the two long span, low steel beams ($pVd/M_{max} = 0.0025$) containing Aggregates 5 and 7. Estimated values of v_c for these beams are shown in the upper left diagram. The other aggregates included in the study show similar sequence of tensile resistance ranging from low to intermediate magnitudes.

The data of each diagram of Fig. 9 appear to be quite linear in the relationship of unit shear to the split-cylinder tensile strength and further, by extrapolation, the relationship could quite easily pass through the point of zero values of v_c and f'_{sp}. Consequently, a function of the type

$$v_c = \alpha_2 f'_{sp} \quad (5)$$

was assigned to each of the five diagrams. Examination of the five different slopes, α_2, indicated that the change of slope was a function of the beam characteristics, pVd/M_{max}, and that this function could be fitted almost exactly by a semilogarithmic function. Consequently, a least squares computation was made of the data from 54 beams and the following relation was established:

$$v_c = \frac{V_{c_1}}{bd} = \left(1.09 + 0.292 \text{ Log } \frac{pVd}{M_{max}}\right) f'_{sp} \quad (6)$$

This function is plotted as the dashed lines on Fig. 9. It should be noted that the parameter of beam characteristics, pVd/M_{max}, in this equation differs from a similar parameter of the Committee 326 formula presented in the last section of this report [see Eq. (10)]. From the rather precise agreement of the diagonal tension strengths of Types 2 and 3 beams, shown in Table 5 and the lack of definite location of the diagonal cracks of Fig. 7, it appears that term, M_{max}/V, reflecting the complete shear span, more aptly reflects the behavior of the beams of this program. The term, M/V, of the Committee 326 formula presumes the location of the section of diagonal cracking. Such an assumption is necessary for the analysis of beams with other types of loading and conditions of restraint.

The fit of the semilogarithmic function, Eq. (6), is fortunately best for the longer span beams. These are more representative of beams in service than are the shorter beams. Such closer fit for the longer span beams may reflect a larger proportional effect of principal tensile resistance in these beams. It may be that the rather wide dispersion of points for the Type 1 beams, $pVd/M_{max} = 0.01$ and 0.02, is the result of greater shear stress effect on the diagonal tension resistance.

The over-all coefficient of variation of V_{test}/V_{calc} from Eq. (6) was 8.4 percent. The coefficient of variation for the longest beams with lowest amount of steel, $pVd/M_{max} = 0.0025$, was reduced to 6.7 percent. Thus rather good correlation is shown between the average split-cylinder tensile strength and the unit shear strength of the beams at diagonal cracking. Such good correlation over the wide range of beam characteristics (pVd/M_{max} varying from 0.0025 to 0.02) and the wide range of

compressive strengths (3000 to 9000 psi) strongly indicates that the split-cylinder test may be used as a reliable measure of unit shear strength.

Deflections

The midspan deflection of all beams was measured at each increment of load up to the point of initial diagonal cracking. The measured deflections of all new beams prior to diagonal cracking are reported in Table 6. Computed deflection at the same loading are also given in the table along with the moduli of elasticity of steel and concrete and the

TABLE 6 — BEAM PROPERTIES AND DEFLECTION COMPARISON

Beam No.	E_c,* 10^6 psi	E_s, 10^6 psi	n	p	Computed k	I,† in.⁴	Computed center deflection, in.	Measured center deflection, in.
Applied load = 10 kips								
3A4	2.00	27.75	13.9	0.0125	0.44	575	0.305	0.305
4A4	1.54	27.75	15.4	0.0125	0.46	616	0.368	0.350
10A4	1.90	27.75	14.6	0.0125	0.45	595	0.308	0.309
8A4	3.21	27.75	8.6	0.0125	0.37	415	0.264	0.294
2B4	1.69	27.75	16.4	0.0125	0.47	641	0.325	0.318
3B4	1.80	27.75	15.4	0.0125	0.46	616	0.318	0.300
4B4	1.80	27.75	15.4	0.0125	0.46	616	0.318	0.332
5B4	2.30	27.75	12.1	0.0125	0.42	524	0.293	0.286
6B4	2.70	27.75	10.0	0.0125	0.39	460	0.278	0.292
7B4	2.16	27.75	12.8	0.0125	0.43	546	0.298	0.288
10B4	2.06	27.75	13.5	0.0125	0.44	564	0.304	0.290
13B4	1.97	27.75	14.1	0.0125	0.44	580	0.308	0.279
8B4	3.48	27.75	8.0	0.0125	0.36	391	0.257	0.257
10BW4	2.15	27.75	12.9	0.0125	0.43	548	0.297	0.277
8BW4	3.46	27.75	8.0	0.0125	0.36	393	0.257	0.219
Applied load = 12 kips								
2B3	1.60	27.75	17.3	0.0125	0.48	663	0.096	0.096
3B3	1.79	27.75	15.5	0.0125	0.46	618	0.092	0.087
4B3	1.80	27.75	15.4	0.0125	0.46	616	0.092	0.090
5B3	2.20	27.75	12.6	0.0125	0.43	540	0.086	0.081
6B3	2.78	27.75	10.0	0.0125	0.39	460	0.080	0.087
7B3	2.13	27.75	13.0	0.0125	0.43	551	0.086	0.072
10B3	2.14	27.75	13.0	0.0125	0.43	350	0.086	0.080
13B3	1.94	27.75	14.3	0.0125	0.45	587	0.090	0.091
8B3	3.43	27.75	8.1	0.0125	0.36	396	0.075	0.075
Applied load = 14 kips								
2B2	1.62	28.2	17.4	0.025	0.59	984	0.310	0.330
3B2	1.87	28.2	15.1	0.025	0.57	913	0.328	0.335
4B2	1.80	28.2	15.7	0.025	0.58	932	0.293	0.313
5B2	2.21	28.2	12.8	0.025	0.54	833	0.268	0.273
6B2	2.68	28.2	10.5	0.025	0.51	746	0.247	0.261
7B2	2.18	28.2	12.9	0.025	0.54	840	0.268	0.275
10B2	2.04	28.2	13.8	0.025	0.55	872	0.278	0.292
13B2	1.92	28.2	14.7	0.025	0.57	900	0.285	0.287
8B2	3.51	28.2	8.0	0.025	0.46	632	0.225	0.228
Applied load = 16 kips								
7B1X	2.20	27.25	12.4	0.025	0.54	820	0.075	0.075
10B1	2.07	28.2	13.6	0.025	0.55	864	0.075	0.085
13B1	1.88	28.2	15.0	0.025	0.57	910	0.079	0.081

*Secant modulus of elasticity at 0.3 f_c'.
†Moment of inertia of cracked transformed cross section

ratios n, p, and k. The deflections were computed by the "cracked transformed cross section" method. The ratio of depth to neutral axis to effective depth, k, was computed by the standard formula

$$k = [2np + (np)^2]^{1/2} - np \quad (7)$$

With this formula and the parallel axis theorem, a convenient expression can be derived for the moment of inertia, I, of the cracked cross section. This expression is:

$$I = \frac{bd^3}{6} k^2(3 - k) \quad (8)$$

Examination of Table 6 and of the corresponding data of the earlier report[1] shows that the cracked cross section method of deflection computation is quite adequate for the beams containing 2.5 percent of tension reinforcement. The measured deflections of these beams agree quite closely with that computed for all loadings nearly to the diagonal cracking load. The measured load deflection diagrams for the beams containing 1.25 percent reinforcing steel were considerably more curvilinear than the diagrams for the beams with heavier steel. The computed deflections of the beams with lighter steel were generally larger than the measured deflections in the working stress ranges, and the curve of measured deflection crossed the computed deflection line at 70 to 100 percent of the diagonal cracking load. At 50 percent of the cracking load, the measured deflections were approximately 90 percent of the computed deflections. The comparison of computed and measured deflections indicates that the cracked cross section method offers an adequate and conservative method for prediction of deflection in lightweight structures.

Comparison of the measured deflections of the lightweight beams and the normal weight beams of the same compressive strength, span and tensile reinforcement confirms the findings of the earlier study. For example, if the deflections of beams containing Aggregate 4 are compared to those of the corresponding beams containing Aggregate 8, it is found that the deflections of the lightweight beams are only 15 to 35 percent greater than those of the normal weight beams, even though the modulus of elasticity of Aggregate 4 concrete is only 50 percent of that for the concrete containing the sand and gravel, Aggregate 8.

PROPOSED ULTIMATE LOAD DESIGN RECOMMENDATIONS FOR SHEAR IN LIGHTWEIGHT CONCRETE BEAMS

In addition to the beams of this Portland Cement Association investigation, the University of Texas[3] has made the results of an excellent series of tests on lightweight concrete available to ACI Committee 213. The following considerations of diagonal tension resistance of lightweight concrete are based on the Texas study as well as that at the PCA labo-

TABLE 7 — PROGRAM OF BEAM TESTS AT THE UNIVERSITY OF TEXAS

No. of Beams	Shear span, in.	a/d	Tension steel, percent	Compression steel	Nominal f_c', psi
4	24 to 43.2	3.0 to 5.4	0.75	1-#4	3800
4*	18 to 42	3.0 to 7.0	1.48	1-#4	3800
4	10.8 to 40	1.35 to 5.0	3.03†	1-#6	3500
6	32	4.0	0.75 to 5.38	1-#6	3500
5	32	4.0	3.03	1-#6	2800 to 6800
3	24 to 40	3.0 to 5.0	0.91	2-#4	4500
1	32	4.0	2.73	2-#7	4500
27 = total lightweight beams					

All beams unless noted: $b = 5.5$ in., $d = 8.0$ in.
All beams tested at 7 days after 6 days moist curing, except the last four which were moist cured 2 or 3 days and tested dry at 135 to 142 days.
*Beams where $b = 4.5$ in., $d = 6$ in.
†Hard grade steel.

ratories. Examination of the test results of two other investigations have been made and these studies provide good corroboration of the design recommendations made in this report.

The 27 lightweight concrete beams tested at the University of Texas were fabricated with concrete containing a single aggregate and generally with compressive strengths in the range of 3500 to 4000 psi. All beams contained compression steel. The program was designed to investigate the effects of varying shear spans and percentages of tension steel. Particular emphasis was placed on large shear spans and low steel percentages. The total program was separated into six parts as follows:

Three parts studied the effect of variable shear span, each at a different level of tensile reinforcement. A fourth portion held the shear span constant but varied the longitudinal steel over a wide range. The fifth employed beams of the same span and steel to study the effect of variable compressive strength. The sixth was particularly concerned with the effect of long-time drying on diagonal tension resistance. The program of tests of lightweight beams at the University of Texas is given in Table 7.

Design equation for lightweight concrete

Comparison of lightweight beams and normal weight beams of similar design shows that the general behavior of the lightweight material is similar to that of normal weight concrete. The differences that develop are those in magnitude of diagonal tension resistance and not in stress distribution. As with stone concrete, beam collapse may occur at the formation of the diagonal crack, or redistribution of forces may be accomplished with ultimate failure occurring under shear-compression. The lightweight tests give further justification to the principle expressed by the Committee 326 report (i.e., only diagonal cracking load should be considered as the ultimate load in design of non-web-reinforced members). This compatible behavior of normal weight and lightweight beams

justifies the development at this time of an ultimate load design formula for the diagonal tension resistance of lightweight concrete.

As Bower and Viest[14] have pointed out, several earlier studies show that the magnitude of shear at initial diagonal tension cracking is a function of several variables. The most important of these are strength of concrete, percentage of tension reinforcement, beam dimensions, and ratio of moment to shear. The above authors have suggested that the shear in non-web-reinforced, normal weight concrete beams may be related to the above variables by the formula

$$\frac{v_c}{\sqrt{f_c'}} = \frac{V}{bd\sqrt{f_c'}} = 1.9 + 2500\frac{pd}{\frac{M}{V}\sqrt{f_c'}} \leq 3.5 \qquad (9)$$

where V is the total external shear and M/V is the ratio of moment to shear defined by the following equation:

$$\frac{M}{V} = \frac{M_{max}}{V} - d \geq \frac{M_{max}}{V} - \frac{a}{2} \qquad (10)$$

ACI-ASCE Committee 326, Shear and Diagonal Tension, has concurred in this empirical formula, and their notation and numerical values of constants are as shown. For convenient reference below, the term $V/bd\sqrt{f_c'}$ is called Parameter A, and the term $pVd/M\sqrt{f_c'}$ is referred to as Parameter B. It should be noted that this definition of Parameter B is the reciprocal of that adopted by the Committee 326 report.

A study of the diagonal tension tests of lightweight concrete from the two sources clearly indicates that the variables of Eq. (9) are also the important variables in design of lightweight concrete. Further, this study shows that these variables can be grouped into the same parameters as for normal weight concrete.

It is logical then to assume that the trend of lightweight concrete data may be expressed in an empirical form similar to Eq. (9). The numerical

TABLE 8 — CONSTANTS DETERMINED BY MULTIPLE CORRELATION FOR LIGHTWEIGHT CONCRETE DIAGONAL TENSION FORMULA*

Test series	C_1	C_2	No. of beams	Coefficient of variation, percent
University of Texas	1.11	6940	26	14.7
PCA Aggregate 2	1.15	4360	5	8.5
PCA Aggregate 3	1.30	4820	6	6.8
PCA Aggregate 10	1.37	4130	6	9.9
PCA Aggregate 6	1.39	4090	5	6.9
PCA Aggregate 13	1.41	2800	4	5.4
PCA Aggregate 4	1.42	2870	8	10.7
PCA Aggregate 7	1.90	5560	6	2.4
PCA Aggregate 5	1.93	2970	4	4.9

* $v_c = C_1\sqrt{f_c'} + C_2\frac{pVd}{M}$

TABLE 9 — PCA LIGHTWEIGHT SIMPLY SUPPORTED BEAMS WITH TWO SYMMETRICAL CONCENTRATED LOADS

Beam No.	$\frac{1000pVd}{M\sqrt{f_c'}}$	$\frac{V_{calc}}{bd\sqrt{f_c'}}$	$\frac{V_{test}}{bd\sqrt{f_c'}}$	$\frac{V_{test}}{V_{calc}}$
Aggregate 2—Expanded shale				
2B4	0.04439	1.266	1.231	0.972
2B2	0.08946	1.435	1.703	1.187
2B3	0.12048	1.552	1.711	1.102
2A1	0.27481	2.131	2.512	1.179
2B1	0.22791	1.955	1.948	0.996
	$n = 5$		Avg	1.087
			C.V.*	8.7
Aggregate 3—Expanded shale				
3A4	0.05028	1.289	1.534	1.190
3B4	0.04704	1.276	1.382	1.083
3B2	0.09276	1.448	1.885	1.302
3B3	0.12899	1.584	1.967	1.242
3A1	0.28925	2.185	2.731	1.250
3B1	0.26057	2.078	2.460	1.184
	$n = 6$		Avg	1.209
			C.V.*	7.2
Aggregate 4—Expanded clay				
4A4	0.05603	1.310	1.668	1.273
4B4	0.04361	1.264	1.331	1.053
4B2	0.08752	1.428	1.778	1.245
4B3	0.11757	1.541	1.557	1.010
4A1	0.30536	2.245	2.680	1.194
4B1	0.23940	1.998	2.447	1.225
4C1	0.39835	2.594	2.315	0.892
4D1	0.36898	2.484	2.481	0.999
	$n = 8$		Avg	1.111
			C.V.*	8.9
Aggregate 5—Expanded slate				
5B4	0.04408	1.265	—	—
5B2	0.08902	1.434	2.238	1.561
5B3	0.11900	1.546	2.267	1.466
5A1	0.28216	2.158	2.918	1.352
5B1	0.24086	2.003	2.511	1.254
	$n = 4$		Avg	1.408
			C.V.*	10.7
Aggregate 6—Expanded slag				
6B4	0.04480	1.268	1.482	1.169
6B2	0.08839	1.431	1.908	1.333
6B3	0.11998	1.550	1.920	1.239
6A1	0.27517	2.132	2.588	1.214
6B1	0.23886	1.996	2.215	1.108
	$n = 5$		Avg	1.213
			C.V.*	8.0
Aggregate 7—Expanded shale				
7B4	0.04476	1.268	—	—
7B2	0.08874	1.433	2.479	1.730
7B3	0.11936	1.548	2.485	1.605
7A1X	0.29421	2.203	3.567	1.619
7A1	0.25599	2.060	3.378	1.640
7B1X	0.24368	2.014	3.306	1.642
7B1	0.23117	1.967	3.086	1.569
	$n = 6$		Avg	1.634
			C.V.*	3.9
Aggregate 9—Experimental Expanded shale				
9C1	0.40094	2.604	2.482	0.953
Aggregate 10—Expanded shale				
10A4	0.05456	1.305	1.791	1.372
10B4	0.04480	1.268	1.413	1.114
10BW4	0.04635	1.274	1.368	1.074
10B2	0.09390	1.452	1.985	1.367
10B3	0.11757	1.541	1.904	1.236
10B1	0.23913	1.997	2.277	1.140
	$n = 6$		Avg	1.217
			C.V.*	9.8
Aggregate 13—Expanded shale				
13B4	0.04348	1.263	1.460	1.157
13B2	0.08787	1.430	1.673	1.170
13B3	0.11757	1.541	1.838	1.193
13B1	0.23716	1.989	2.033	1.022
	$n = 4$		Avg	1.136
			C.V.*	7.4

*Coefficient of variation, percent.

$$\frac{V_{calc}}{bd\sqrt{f_c'}} = 1.1 + 3750\,\frac{pVd}{M\sqrt{f_c'}} \leqq 3.5$$

$$\frac{M}{V} = \frac{M_{max}}{V} - d \geqq \frac{M_{max}}{V} - \frac{a}{2}$$

constants, however, will be changed to reflect the lowered tensile resistance of some lightweight concretes. It is also assumed that the location of the diagonal crack and consequently the proper ratio of M/V for analysis of lightweight beams will be defined identically with that for normal weight concrete as in Eq. (10). The type form of the design equation for lightweight concrete is then

$$\frac{v_u}{\sqrt{f_c'}} = \frac{V}{bd\sqrt{f_c'}} = C_1 + C_2 \frac{pVd}{M\sqrt{f_c'}} \quad \text{.....(11)}$$

In view of the characteristic levels of tensile resistance discussed previously, values of the constants C_1 and C_2 have been determined on a multiple correlation basis for each of the concretes containing the various aggregates. It was found that for the aggregates included in the two test sources, C_1 varied from 1.11 to 1.93. In other words C_1 varied from a low value up to the value 1.9 assigned by Committee 326 to normal weight concrete, depending on the aggregate considered. Meanwhile, the constant C_2 varied in a haphazard manner from a low value of 2800 to a high of 6940. For most of the aggregates, C_2 was 4000 or higher. It appears then that lightweight concrete, in relation to normal weight concrete, tends to have a lower initial point (at a value of Parameter B equal to zero) and a more rapid increase in Parameter A, as Parameter B increases. These "best fit" values of C_1 and C_2 are shown in Table 8.

The original PCA test series demonstrated that lightweight concrete fails in the same range of unit shear strength as does the normal weight concrete when the value of Parameter B is quite large. This conclusion is also confirmed by the appropriate tests included in the University of Texas investigation. In view of these data and the higher trend of constant C_2 for lightweight concrete, it is logical, and convenient, to assume an upper limit of Parameter A at 3.5, the same limitation imposed by Committee 326 for stone concrete. By the same reasoning, this limitation will also be assumed to occur at the same point as for normal weight concrete, that is at the value of Parameter B equal to 0.00064. This value is obtained by equating Eq. (9) to 3.5 and solving for Parameter B. Thus, one point on the design function for lightweight diagonal tension has been obtained.

Certainly the design formula chosen for lightweight beams should be conservative. Magnitudes of unit shear derived from the formula should reflect that the characteristics of particular lightweight aggregates vary widely and that only a few of the aggregates available on the market have been used in tests for diagonal tension. On the other hand, examination of the existing test data reveals that some aggregates develop diagonal tension strength approaching that of the stone concretes. In the interest of economical design these aggregates should not be unduly penalized.

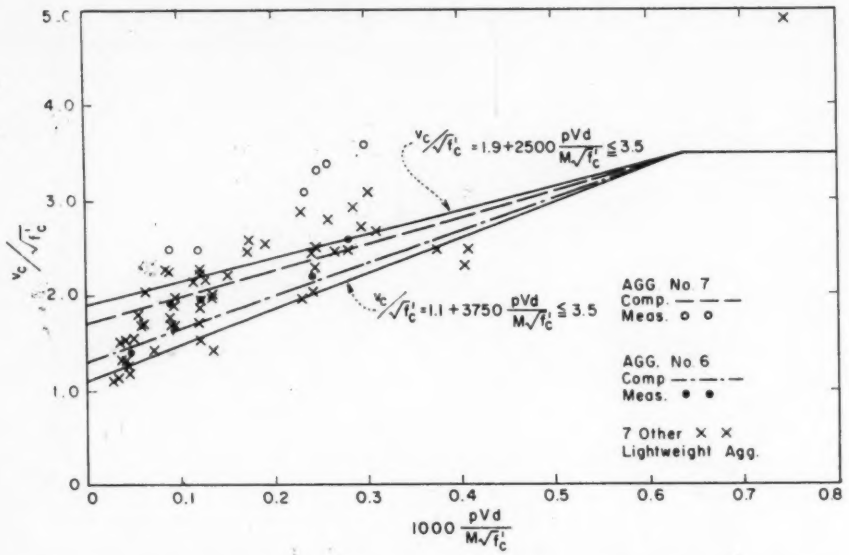


Fig. 10—Design equations for lightweight beams without web reinforcement

It appears from the tabulation of C_1 in Table 8 that a value of Parameter A, for Parameter B equal to zero, may be chosen as 1.1. A second point for a *minimum* lightweight design formula is now available along with that chosen above at Parameter A equal to 3.5 when Parameter B equals 0.00064. Substitution of these simultaneous values in the typical form Eq. (11), results in the following design formula:

$$\frac{v_c}{\sqrt{f_c'}} = \frac{V}{bd\sqrt{f_c'}} = 1.1 + 3750\frac{pVd}{M\sqrt{f_c'}} \leq 3.5 \quad \text{.....(12)}$$

Provided that

$$\frac{M}{V} = \frac{M_{max}}{V} - d \geq \frac{M_{max}}{V} - \frac{a}{2} \quad \text{.....(10)}$$

Fig. 10 shows the trend of the data from the 72 lightweight beams available and the relation of this trend to Eq. (12). Detailed computations, Tables 9 and 10, have been made of the relative safety for each of the beams tested when compared to Eq. (12). Table 11 summarizes the average values of V_{test}/V_{calc} and the coefficients of variation for each aggregate series from these detailed computations. Examination of the individual values of V_{test}/V_{calc} shows that 14 percent of the measured values fall on or below the line defined by Eq. (12). This is practically the same as the percentage of those normal weight beams having the same range of Parameter B, which fall on or below the line defined by Eq. (9), as reported by Committee 326.

Examination of Fig. 10 indicates the excessive safety of design Eq. (12) in regard to several aggregates. This equation then should be considered applicable only to those aggregates that exhibit low diagonal tension resistance or in cases where sufficient laboratory data are lacking for upgrading the coefficients of the design formula. A convenient method for providing safe but reasonable values for the unit shear for any particular aggregate now follows.

Design equations for concretes containing a particular lightweight aggregate

The limitation of 3.5 for Parameter A provides a conservative magnitude of unit shear for beams of the short, deep variety. Further, the requirement that this limitation for lightweight concrete should occur at the same point (Parameter $B = 0.00064$) as specified for normal weight concrete, provides further assurance of safe values. This point, then, should remain constant in the design formula associated with any particular aggregate. A second point established for concrete made with this particular aggregate would allow computation of the empirical constants associated with Parameters A and B by simple solution of simultaneous equations. This second point should be established for small values of Parameter B such as occur in most service beams. The recent investigations provide a simple but safe test procedure for determination of this second point and the specific constants for the special design formula. The relationship between unit shear stress and split-cylinder tension, exemplified by Eq. (6), provides the "tool" for obtaining the special formula.

The upper left diagram of Fig. 9 represents the relationship between v_c and f'_{sp} for the longest beams with the smallest amount of tensile steel for which adequate tension tests are available. It is reasonable, therefore, to specify an empirical relationship between these two variables for this particular ratio of beam physical characteristics. The dashed line on the figure, Eq. (6), represents a "best fit" of all the data. By examination of this function for all 54 beams, it was found that if the slope of the function was reduced by 10 percent, all lightweight beam tests except one would exceed the reduced function. The unit shear of this particular beam falls below the reduced function by only 5 psi. The solid line of Fig. 9 (upper left diagram) represents the reduced function and is expressed by the empirical relationship

$$v_c = 0.3 f'_{sp} \qquad (13)$$

provided $pVd/M = 0.00312$, where $pVd/M_{max} = 0.0025$ [see Eq. (10)].

Application of the limitation of Parameter A to 3.5 and of Eq. (13) to the type formula, Eq. (11), results in two simultaneous equations as

follows:

$$C_3 + 0.00064\,C_4 = 3.5$$

and

$$C_3 + 0.00312\,\frac{C_4}{\sqrt{f_c'}} = 0.3\,\frac{f'_{sp}}{\sqrt{f_c'}}$$

Subtraction of the lower equation from the upper yields

$$C_4\left(0.00064 - \frac{0.00312}{\sqrt{f_c'}}\right) = 3.5 - \frac{0.3\,f'_{sp}}{\sqrt{f_c'}}$$

TABLE 10— UNIVERSITY OF TEXAS LIGHTWEIGHT BEAMS WITH TWO SYMMETRICAL CONCENTRATED LOADS

$$\frac{V_{calc}}{bd\sqrt{f_c'}} = 1.1 + 3750\,\frac{pVd}{M\sqrt{f_c'}} \leq 3.5$$

Beam No.	$\frac{1000\,pVd}{M\sqrt{f_c'}}$	$\frac{V_{calc}}{bd\sqrt{f_c'}}$	$\frac{V_{test}}{bd\sqrt{f_c'}}$	$\frac{V_{test}}{V_{calc}}$
Expanded shale aggregate				
A4	0.02791	1.205	1.120	0.929
A1	0.03577	1.234	1.329	1.077
T3	0.03376	1.227	1.145	0.933
Gu1	0.03816	1.243	1.511	1.216
Go5	0.04132	1.255	1.514	1.206
A2	0.04645	1.274	1.513	1.188
T2	0.04355	1.263	1.176	0.931
Gu2	0.06471	1.343	2.027	1.509
A3	0.06100	1.329	1.700	1.279
Go4	0.08298	1.411	2.273	1.611
T1	0.07063	1.365	1.447	1.060
Gu3	0.09654	1.462	1.722	1.178
Go3	0.11420	1.528	2.167	1.418
Gu4	0.11941	1.548	2.175	1.405
Ma3	0.12830	1.581	2.173	1.374
T4	0.13462	1.605	1.413	0.880
Ma1	0.17295	1.749	2.587	1.479
E11	0.19191	1.820	2.548	1.400
E12	0.17271	1.748	2.449	1.401
E13	0.14956	1.661	2.221	1.337
E14	0.13521	1.607	2.008	1.250
E15	0.12222	1.558	1.540	0.988
Go6	0.17100	1.741	2.464	1.415
Go1	0.22973	1.961	2.889	1.473
Ma2	0.25906	2.071	2.798	1.351
Go2	0.30308	2.237	3.081	1.377
Ma4	0.74605	3.500	4.873	1.392
	$n = 27$		Average =	1.261
			Coefficient of variation, percent =	15.90

For the extreme ranges of compressive strength from 2000 to 9000 psi, the second term inside the parentheses varies only from 0.00007 to 0.00003. Therefore, the value inside the parentheses can be closely approximated as 0.0006 and the above formula may be expressed as:

$$\frac{f'_{sp}}{\sqrt{f_c'}} = 11.67 - 0.002\,C_4 \quad \text{...(14a)}$$

or, if preferred,

$$\frac{f'_{sp}}{\sqrt{f_c'}} = 0.729 + 3.125\,C_3 \quad \text{...(14b)}$$

TABLE 11—SUMMARY OF AVERAGE V_{test}/V_{calc} FOR VARIOUS LIGHTWEIGHT AGGREGATES

From formula $v_c = 1.1\sqrt{f_c'} + 3750\,\frac{pVd}{M}$

Test series, aggregate	No. of beams	Average $\frac{V_{test}}{V_{calc}}$	Coefficient of variation, percent
PCA 2	5	1.087	8.5
PCA 4	8	1.111	8.9
PCA 13	4	1.136	7.4
PCA 3	6	1.209	7.2
PCA 6	5	1.213	8.0
PCA 10	6	1.217	9.8
Univ. of Texas	27	1.261	15.9
PCA 5	4	1.408	10.7
PCA 7	6	1.634	3.9
All beams	71*	1.253	15.9

*PCA Aggregate 9 beam not included in tabulation.

These are simultaneous linear relationships between $f'_{sp}/\sqrt{f_c'}$ and the constants associated with Parameters A and B for concrete with any particular aggregate.

The design formula for any particular aggregate may thus be established on the basis of adequate compression and split-cylinder tests. The cylinders for these tests should be fabricated, cured, and tested in accordance with the provisions presented later in this report under "Recommendations for Diagonal Tension Ultimate Load Design of Lightweight Concrete Beams." The test results then allow the determination of the ratio $f'_{sp}/\sqrt{f_c'}$ and consequently, from Eq. (14a) and (14b), the values of C_3 and C_4 for the particular aggregate. These values have been tabulated in the recommendations over a range of this ratio from that corresponding to the minimum formula, Eq. (12), to that assigned to normal weight concrete, Eq. (9).

Examples of upgraded design formula

Two examples have been computed to demonstrate the proposed method by which the design formula for particular lightweight beams may be established by test. The pertinent data for these computations are presented in Table 12 together with the resulting design formula constants taken from the recommended table.

Fig. 10 shows these upgraded design formulas relative to the test data and also relative to the minimum lightweight concrete design formula and to that for normal weight concrete from Eq. (9). Table 13 summarizes the new average values of V_{test}/V_{calc} for the example aggregates and compares them with the averages computed by the minimum design Eq. (12).

TABLE 12 — DATA FOR DESIGN CONSTANTS OF EXAMPLE AGGREGATES

Aggregate No.	f_c', psi	f'_{sp}, psi	f_c' avg, psi	f'_{sp} avg, psi	$\frac{f'_{sp\ avg}}{\sqrt{f_c'\ avg}}$	C_3	C_4
6	3320 4890	302 327	4105	314	4.90	1.3	3440
7	3280 4840	348 408	4060	378	5.93	1.7	2810

TABLE 13 — COMPARISON OF V_{test}/V_{calc} AVERAGES WITH THOSE COMPUTED BY MINIMUM DESIGN FORMULA

Test series	No. of beams	Upgraded formula		Minimum formula	
		Average $\frac{V_{test}}{V_{calc}}$	Coefficient of variation, percent	Average $\frac{V_{test}}{V_{calc}}$	Coefficient of variation, percent
PCA Aggregate 6	5	1.105	6.7	1.213	8.0
PCA Aggregate 7	6	1.333	5.7	1.634	3.9

TABLE 14 — DESIGN FORMULAS FOR PARTICULAR AGGREGATES RESULTING FROM CYLINDER SPLITTING AND COMPRESSION TESTS

Beam No.	$\frac{1000\,pVd}{M\sqrt{f_c'}}$	$\frac{V_{calc}}{bd\sqrt{f_c'}}$	$\frac{V_{test}}{bd\sqrt{f_c'}}$	$\frac{V_{test}}{V_{calc}}$
Aggregate 6—Expanded slag				
$\frac{V_{calc}}{bd\sqrt{f_c'}} = 1.30 + 3438\,\frac{pVd}{M\sqrt{f_c'}} \leq 3.5$		$\frac{M}{V} = \frac{M_{max}}{V} - d \geq \frac{M_{max}}{V} - \frac{a}{2}$		
6B4	0.04480	1.454	1.482	1.019
6B2	0.08839	1.604	1.908	1.190
6B3	0.11998	1.712	1.920	1.121
6A1	0.27517	2.246	2.588	1.152
6B1	0.23886	2.121	2.215	1.044
$n = 5$			Average	1.105
			Coefficient of variation, percent	6.7
Aggregate 7—Expanded shale				
$\frac{V_{calc}}{bd\sqrt{f_c'}} = 1.70 + 2812\,\frac{pVd}{M\sqrt{f_c'}} \leq 3.5$		$\frac{M}{V} = \frac{M_{max}}{V} - d \geq \frac{M_{max}}{V} - \frac{a}{2}$		
7B4	0.04476	1.826	—	—
7B2	0.08874	1.950	2.479	1.271
7B3	0.11936	2.036	2.485	1.221
7A1X	0.29421	2.527	3.567	1.412
7A1	0.25599	2.420	3.378	1.396
7B1X	0.24368	2.385	3.306	1.386
7B1	0.23117	2.350	3.086	1.313
$n = 6$			Average	1.333
			Coefficient of variation, percent	5.7

TABLE 15 — DESIGN CONSTANTS

Range of ratio $f'_{sp}/\sqrt{f_c'}$	Constant, C_3	Constant, C_4	Constant, C_5
4.01-4.31	1.1	3750	4280
4.32-4.63	1.2	3590	4110
4.64-4.94	1.3	3440	3930
4.95-5.25	1.4	3280	3750
5.26-5.56	1.5	3120	3570
5.57-5.88	1.6	2970	3390
5.89-6.19	1.7	2810	3210
6.20-6.50	1.8	2660	3040
6.51-6.67	1.9	2500	2850

The excessive safety factors for these two aggregates have been reduced to reasonable but still safe magnitudes. The detailed computation of V_{test}/V_{calc} for the example beams is given in Table 14.

Two major considerations which influence the choice of a diagonal tension design formula are: first, the equation should be simple to facilitate everyday design work; secondly, and more importantly, the equation should be such that the ultimate strength of beams resulting from practical design will be governed by flexure rather than by shear. These important considerations are satisfied by the proposed method of diagonal tension design for lightweight concrete. The formula is the same type as that for normal weight and the constants in the lightweight concrete method have been chosen by an even more conservative method. The slopes for the lightweight concretes have been deliberately lowered by setting the upper limit of unit shear at $3.5\sqrt{f_c'}$ and the magnitudes of the constants, C_3 and C_4, have been effectively reduced by expressing the unit shear stress as $0.3f'_{sp}$ at $pVd/M = 0.00312$. This highly conservative nature of the lightweight concrete method should adequately take care of such elusive effects as shrinkage and continuity on diagonal tension resistance.

As a final stage in the development of the proposed ultimate load design formula for normal weight concrete, Committee 326 transformed Eq. (9) into a form considered more convenient for everyday design practice. This same transformation can be easily applied to the proposed lightweight concrete design formulas by multiplying the constant C_4 by the factor 8/7.

SUMMARY AND CONCLUSIONS

1. This investigation was limited to simply supported beams with two symmetrical concentrated loads. With combinations of two spans, 10 ft and 6 ft 6 in., two amounts of longitudinal reinforcement (1.25 and 2.5 percent), ten different aggregates and, for most conditions, two levels

of compressive strength, single beam tests were generally used for each condition.

2. The longer span beams generally failed completely at formation of the initial diagonal crack. The ability of the shorter span beams to achieve stress redistribution after diagonal cracking was materially affected by chance location of the crack. Therefore, the load at diagonal cracking should be considered the ultimate load for non-web-reinforced concrete beams.

3. Each beam test was accompanied with 6 x 12-in. cylinders for compressive and tensile splitting tests. The tensile splitting test provides an adequate measure of the relative tensile strength of concrete provided that sufficient samples are tested to give a reliable average.

4. Drying at 50 percent relative humidity will reduce the split cylinder tensile strength of lightweight concretes by magnitudes of 0 to 40 percent. The amount of tensile strength reduction will depend on the particular aggregate and on the compressve strength of the concrete.

5. A good correlation was found between the nominal unit shear strength of the beams and the accompanying split-cylinder tensile strengths of dry concretes (cured and dried according to ASTM C 330-59T). This is to be expected since it has been recognized that diagonal tension resistance is a primary function of tensile strength. Therefore, the split-cylinder tension test may be used as a reliable measure of unit shear strength.

6. The split-cylinder tests and the beam tests have shown that the nominal unit shear strength of concrete containing a particular lightweight aggregate is determined by the characteristic level of tensile strength associated with that aggregate. The unit shear strengths of the lightweight concrete beams varied from approximately 60 to 100 percent of the unit shear strength of comparable beams containing normal weight aggregate, depending on the lightweight aggregate considered and on the beam characteristics.

7. Comparison of the unit shear strengths at diagonal cracking with the ACI Building Code working stresses reveals that inadequate factors of safety existed for the lightweight concrete beams with longer spans and lower steel percentages. Thus, code provisions in regard to diagonal tension resistance of structural lightweight concrete should be established at the earliest opportunity. These provisions can probably be best established through ultimate load design criteria. These criteria should be conservative to reflect the increased chance of higher shrinkage stresses existing in restrained beams.

8. Deflections of both the normal weight and the lightweight beams after flexural cracking under working loads were computed by the transformed cross section method with acceptable and conservative accuracy. The measured deflections have shown that consideration of

deflection as inversely proportional to modulus of elasticity is in error on the conservative side by large magnitudes.

RECOMMENDATIONS FOR DIAGONAL TENSION ULTIMATE LOAD DESIGN OF LIGHTWEIGHT CONCRETE BEAMS

In accordance with the proposed design provisions established by ACI-ASCE Committee 326 for normal weight concrete, ACI Committee 213 recommends the following ultimate strength design procedures for structural lightweight concrete beams without web reinforcement.

1. The diagonal tension strength shall be determined on the basis of the average unit shearing stress v computed by the formula $v = V/bd$, where V = shear force, b = width of member, and d = effective depth to centroid of tension steel.

2. For beams of I- and T-section, or for floor joist construction the web width b' shall be substituted for the width b.

3. The ultimate diagonal tension strength v_c of a lightweight concrete section of an unreinforced web shall be computed by either of the formulas

$$v_c = C_3\sqrt{f_c'} + C_4\frac{pVd}{M}, \quad \text{or} \quad v_c = C_3\sqrt{f_c'}\left(\frac{f_s}{f_s - C_5}\right),$$

where the constants, C_3 and C_4, or C_3 and C_5, shall be selected from Table 15 and

f_c' = concrete compressive strength, psi
f_s = longitudinal steel stress at the section considered, psi
p = ratio of area of flexural tension reinforcement to the effective cross-sectional area of the beam
M/V = the ratio of moment to shear at the section considered

(a) For purposes of using Table 15, the ratio of the split-cylinder tensile strength, f'_{sp}, to the square root of the compressive strength, f_c', shall be determined as follows:

The tests for compressive strength shall be made in accordance with ASTM C 330-59T, which provides for 7 days moist curing followed by storage at 73 F and 50 percent relative humidity until time of test. To conform with Table 15, the test shall be at 28 days. The test cylinders shall be 6 x 12 in. and shall be cast from the same batches of concrete as the cylinders that are to be tested by splitting. A total of twenty-four 6 x 12 in. concrete cylinders shall be cast for both compressive strength and split-cylinder tension, one-half of these at the compressive strength level of approximately 3000 psi and the other half at approximately 5000 psi. The cylinders at each strength level shall be produced from two or more batches. After 7 days moist curing followed by 21 days drying at 73 F, 50 percent relative humidity, four of the test cylinders at each of the two nominal strength levels shall be tested for compressive strength and eight cylinders at each of the two strength levels shall be tested for tension splitting in the manner recommended below. For the purpose of establishing the constants of the design equation from Table 15, the results of all 16 indi-

vidual split-cylinder tests shall be averaged to obtain f'_{sp} and all eight of compression test to obtain f_c' and consequently the ratio of $f'_{sp}/\sqrt{f_c'}$.

To conform with the data from which Table 15 was established, the split-cylinder tests should be performed as follows:

The testing machine shall have an attachment for the head at least 12 in. long to accommodate the 12 in. length of the specimen. Fir plywood pads, also 12 in. long, ⅛-in. thick, shall be inserted between the testing machine platen and the specimen, and between the head and the specimen. The vertical diameter at each end of the cylinder shall be in alignment with the testing machine head center line. The specimen is to be loaded at a rate of 10,000 to 15,000 lb per min until splitting occurs. The split-cylinder tensile strength is then determined from equation $f'_{sp} = P/36\pi$, in which P is the testing machine load at point of vertical diameter splitting.

(b) In the event that adequate test data are lacking, either of the following minimum formulas shall apply:

$$v_c = 1.1\sqrt{f_c'} + 3750\,\frac{pVd}{M} \quad \text{or} \quad v_c = 1.1\sqrt{f_c'}\left(\frac{f_s}{f_s - 4280}\right)$$

(c) The limitation, v_c shall not exceed $3.5\sqrt{f_c'}$, and shall apply to the formulas of either of Paragraphs 3(a) or 3(b).

4. If the ultimate load on the member produces a unit shearing stress v, which exceeds the calculated ultimate diagonal tension strength v_c, web reinforcement must be provided, or the cross section must be increased.

5. The provision of Paragraph 4 is subject to the following limitations depending on the length a of any portion of the member within which the shear diagram retains the same sign: *

(a) If $a > 2d$, the provision of Paragraph 4 shall not apply within the distance d at either end of the length a.

(b) If $2d \geq a \geq \frac{3}{4}\,d$, the provision of Paragraph 4 shall apply only at the section located in the middle of the length a.

(c) If $a < \frac{3}{4}\,d$, the provision of Paragraph 4 shall not apply.

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Resistencia a la tensión y resistencia a la tensión diagonal del concreto estructural liviano

Se describen los ensayos empleados y los resultados obtenidos en una extensión de un estudio anterior sobre la resistencia a la tensión diagonal dado por el autor. Esta extensión del programa original se refiere a las vigas de concreto liviano de tramo más largo y de porcentajes inferiores de acero. Una conclusión importante, es decir, que la carga de agrietamiento diagonal debe considerarse como la carga final para las vigas reforzadas sin alma, ha sido confirmada.

Un gran número de los cilindros de 6 x 12 pulgadas de los concretos de viga fueron rotos por el ensayo de la tensión del "cilíndro partido." Una buena correlación fué establecida entre esta medida indirecta de la tensión y la resistencia al corte de las vigas al agrietamiento diagonal. Esta correlación muestra que la resistencia a la tensión diagonal de los concretos livianos varía de apróximadamente 60 por ciento de la resistencia del concreto semejante de peso normal a casi 100 por ciento, dependiendo de los agregados livianos particulares empleados.

Las recomendaciones propuestas para la carga final de diseño se formulan para el concreto estructural liviano. Estas están de acuerdo general con las recomendaciones del Comisión 326 de ACI-ASCE sobre la tensión al corte y la tensión diagonal para el concreto de peso normal. Se ha verificado que la resistencia a la tensión diagonal de los concretos livianos se ve afectada por las mismas variables que afectan la resistencia del concreto de peso normal. La diferencia entre los dos tipos de materiales es la de la magnitud de la resistencia a la tensión diagonal y no es una diferencia fundamental en el comportamiento.

Las recomendaciones propuestas del diseño también tienen en cuenta las diferencias fundamentales en la resistencia a la tensión, las cuales existen entre los varios agregados livianos. Una combinación de la resistencia a la compresión y el ensayo de la tensión del cilindro partido proporciona una medida conveniente y segura de la resistencia a la tensión diagonal final para ser asociada con cada uno de los varios agregados.

Résistance à la tension et résistance à la tension diagonale de béton léger de construction

Décrit les épreuves utilisées et les résultats obtenus par extension d'une étude antérieure de la résistance á la tension diagonale rapportée par l'auteur. Cette extension du programme initial concerne des poutres de béton léger, de portée plus longue et contenant moins d'acier. Une conclusion intéressante, que la charge de rupture diagonale doit être considerée comme la charge définitive pour les poutres armées sans âme pleine, est confirmêe.

Un très grand nombre de cylindres de 6 x 12 pouces, coulés des bétons dont venaient les poutres, étaient cassés par l'essai de tension dit "Cylindre fendu". Une bonne corrélation était établie, entre cette mesure indirecte de la tension et la résistance au cisaillement des poutres sous la fissuration diagonale. Cette corrélation démontre que la résistance à la tension diagonale de béton léger varie entre 60 pour cent, à peu près, de celle d'un béton analogue à poids normal, et presque 100 pour cent, selon les agrégats légers utilisés.

On présente des essais de préconisations pour la conception définitive des

charges pour le béton léger de construction. Ceux-ci sont en accord général avec les préconisations du Comité 326 de l'ACI-ASCE au sujet de tensions diagonales et de cisaillement pour le béton de poids normal. On a découvert que la résistance à la tension diagonale des bétons légers est gouverné par les mêmes variables qui gouvernent la résistance de béton de poids normal. La différence entre les deux types de matières est une différence d'amplitude de la résistance à la tension diagonale et non pas une différence fondamentale de comportement.

Les essais de préconisation tiennent également en considération les différences fondamentales de résistance à la tension qui existent entre les divers agrégats légers. Une combinaison d'épreuves de la résistance à l'écrasement et de la tension du cylindre fendu fournit une mesure convenable et sûre des résistances définitives à la tension diagonale qui doit être associées à chacun des divers agrégats.

Zugfestigkeit und diagonaler Spannungswiderstand von Leichtbaubeton

Beschreibt die Versuche und Resultate, die in einer Fortsetzung einer frueheren, vom Verfasser berichteten Untersuchung des diagonalen Spannungswiderstandes erhalten wurden. Diese Fortsetzung des urspruenglichen Programms betrifft Leichtbetontraeger mit laengerer Spannweite und nierigerem Stahlgehalt. Eine wichtige Schlussfolgerung, naemlich dass die diagonale Risslast als die aeusserste Last fuer nicht durch Stege verstaerkte Traeger angesehen werden sollten, wurde bestaetigt.

Eine grosse Anzahl von 6 x 12 Zoll-Zylindern aus dem Traegerbeton wurden durch Spannungsversuche mit "Spaltzylindern" gebrochen. Eine gute Beziehung wurde zwischen dieser indirekten Spannungsmessung und der Scherfestigkeit der Traeger bei diagonaler Rissbildung hergestellt. Diese Beziehung zeight, dass der diagonale Spannungswiderstand von Leichtbaubeton von ungefaehr / 60 Prozent des Widerstandes von Normalbeton bis zu fast 100 Prozent reicht, je nach den besonderen Zuschlagstoffen des Leichtbetons, die verwendet werden.

Die vorgeschlagenen aeussersten Belastungen gelten fuer Leichtbaubeton. Diese sind gewoehnlich in Uebereinstimmung den Empfehlungen der ACI ASCE-Auschusses 326 fuer Scher- und Diagonal-Spannung fuer Normalbeton. Es hat sich gezeigt, dass die diagonale Spannungsfestigkeit von Leichtbaubeton von denselben Variablen beeinflusst wird, die auch den Widerstand von Normalbeton beeinflussen. Der Unterschied zwischen den beiden Arten von Material ist ein Groessenunterschied des diagonalen Spannungswiderstandes, und nicht ein fundamentaler Unterschied im Verhalten.

Die vorgeschlagenen Empfehlungen beruecksichtigen also die fundamentalen Unterschiede, die zwischen den verschiedenen Zuschlagstoffen des Leichtbetons bestehen. Eine Verbindung von Druckfestigkeit-und Spaltzylinder-Probe stellt daher eine bequemes und sicheres Mass des aeussersten diagonalen Spannungswiderstandes dar, der jedem der verschiedenen Zuschlagstoffe zugeordnet werden muss.

Title No. 58-2

Prestressed and Precast Concrete Building at Boeing Plant

By ARTHUR R. ANDERSON and A. T. WAIDELICH

Describes the salient design and construction features of a two-story precast and prestressed concrete building at the Developmental Center of the Boeing Airplane Co., Seattle, Wash. The floor area was more than 500,000 sq ft, which was cast, erected, and completed in about 14 months. The story heights are 25 ft each. The column spacing is 60 x 40 ft on the second story and 30 x 40 ft on the first floor, so that the column lengths are alternately 50 and 25 ft.

Details and construction procedures were used which developed full continuity at all joints of the precast members and also achieved load-carrying participation of the roof deck and second floor slab.

The majority of the precast and prestressed members were fabricated on the site, including end-supported prestressed lightweight concrete wall panels as high as 20 ft and spanning a maximum of 40 ft.

■ THE CONCRETE STRUCTURE DESCRIBED HEREIN (see Fig. 1) is a portion of a larger building with a 540 x 350 ft structural steel hangar at one end. In front of this hangar is an airplane apron 540 x 250 ft. This concrete apron is 15 in. thick and its early completion provided an ideal and convenient working area of over 13,000 sq ft which was used as the on-site precasting yard as shown in Fig. 2.

Important considerations in the decision to use a precast and prestressed structure were:

1. A fireproof rating

2. An advanced construction schedule on which an immediate start was possible.

3. The economies possible with a large, paved area adjacent to the building for precast fabrication and the repetition of large numbers of typical structural members

4. The economy of eliminating the extensive 25 ft high form shoring which would have been required for cast-in-place concrete construction.

Under the precast two-story building there were 28,000 sq ft of walkable service tunnels and special vaults. These were cast-in-place concrete, as were the nine stair towers and the two elevator shafts. The remainder of the building frame, the roof, and the walls were precast. The roof deck consisted of 4 x 10 ft precast channel slabs. The second floor was cast-in-place.

DESIGN CRITERIA

The design live load on the roof was 25 lb per sq ft; on the second floor a uniform load of 150 lb per sq ft was used or a concentrated load of 5 kips

In addition all framing members of the roof and second floor were designed for a mechanical equipment load of 5 lb per sq ft plus the concentrated loads of a suspended 5-ton crane at any point in their span.

Lateral loads on the structural frame are wind and the seismic force for Zone 2 as specified in the Pacific Coast Uniform Building Code.

To take the wind and seismic forces, details were developed which firmly tied together all elements to produce a monolithic structure with continuity and rigid joints.

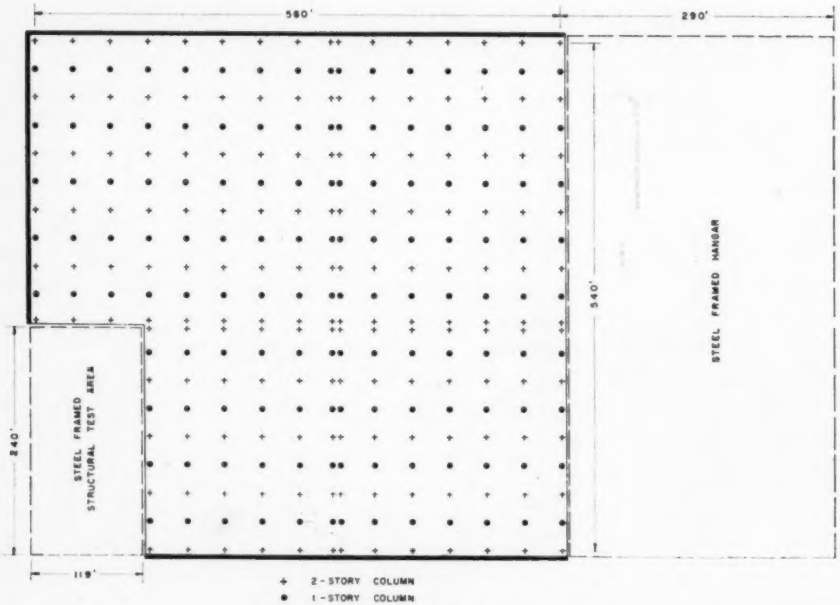

Fig. 1—Basic plan of main building

ACI Director **Arthur R. Anderson** is a partner in Anderson, Birkeland and Anderson, structural engineers, Tacoma, Wash. Dr. Anderson is a specialist in static and dynamic strain measurements, stress analysis, and prestressed construction. During the Boeing project he was in charge of the prestressed design and construction and developed many of the special techniques used on this project. He is currently a member of ACI Committee 324, Precast Reinforced Concrete, Thin Sections, and ACI-ASCE Committee 712, Precast Structural Concrete Design and Construction.

ACI member **A. T. Waidelich** is vice-president for engineering and research, Austin Co., engineers and builders, Cleveland, Ohio. He taught structural engineering at Massachusetts Institute of Technology, Cambridge, and then was assistant professor of civil engineering at Robert College, Istanbul, Turkey, before joining Austin Co. He is especially well qualified to write on the Boeing project since he was in direct charge of design. During 1949-1959, Mr. Waidelich was manager of Austin Co.'s research division and among the projects he supervised were tests of large scale prestressed beams and girders. The tests were among the earliest on full scale specimens.

To accommodate expansion and contraction, the building is separated into four independent structures by two double rows of columns and framing at right angles near the building center lines.

FRAMING SCHEME

Except for the differences occasioned by the two expansion joints, the structure consists of 114 typical 60 x 40 ft two-story elements (Fig. 3). The precast columns are alternately 50 and 25 ft high. In the first story all columns are 24 in. square; in the second story they are 20 in. square.

In the roof the girders span 60 ft and carry 40-ft purlins spaced 10 ft on centers.

In the second floor the girders (running at 90 deg with the girders in the roof) span 40 ft and carry 30-ft beams spaced 10 ft on centers.

All purlins, girders and beams are prestressed. The purlins have a 12 x 32 x 5 in. I-section. The roof girders and the second floor girders have identical 18 x 48 x 6 in. I-sections. See Fig. 4 for a cross section

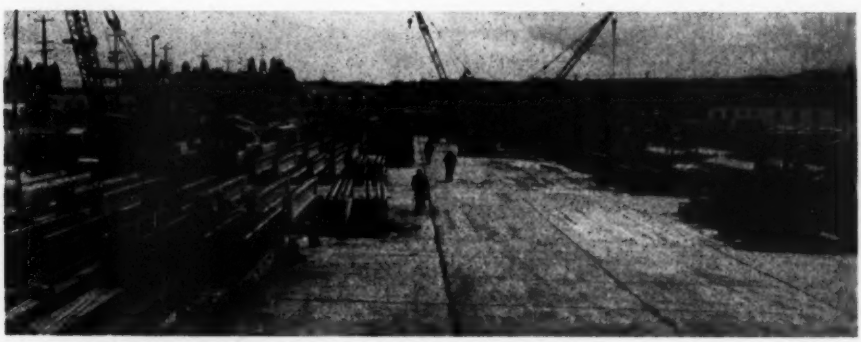

Fig. 2—Precast area; wood forms for columns shown in foreground, steel forms for girders in background

Fig. 3 — North elevation; double plates on columns to support wall panels; shows alternate two- and one-story columns, 60 ft roof girders and 40 ft second floor girders

of a typical roof girder. The second floor beams are 8 x 28 in. in cross section.

CONNECTIONS AND STRUCTURAL CONTINUITY

Column bases (Fig. 5) are a welded assembly of heavy steel angles welded to the main column reinforcement, and have four 1½ in. anchor bolts. Because of this anchorage and the high relative stiffness of the roof and floor framing, the column design assumed that they had fixed ends and points of inflection were taken at the midpoints for lateral loading.

Full continuity was developed at all connections of purlins or beams to girders and at all connections of framing to the columns. A typical interior cross section of building is shown in Fig. 6.

The roof purlins and the second floor beams were designed to act as simple spans for dead load and as continuous spans for live load. For the roof purlins this was accomplished by leaving an 8-in. gap in the roof deck over the purlins for a cast-in-place concrete strip containing negative steel over the supports. Fig. 7 shows typical precast roof panels carried by prestressed purlins. Additional steel placed in the floor slab over the girders provides the negative reinforcement for the floor beams.

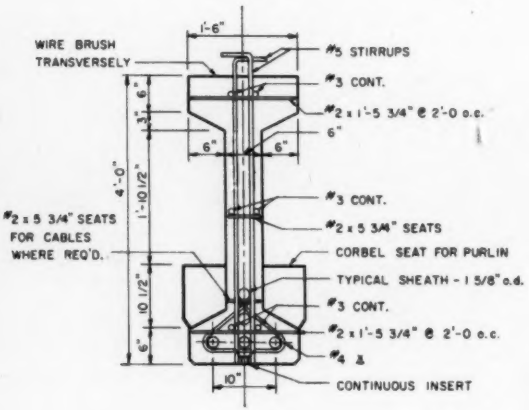

Fig. 4—Cross section of a typical 60 ft roof girder

The second floor girders and the roof girders were designed for composite action and continuity under live load and a portion of the dead load.

After the purlins had been erected a 12 x 12 in.-shore was installed under the midpoint of the roof girder. In placing the 4 x 10 ft roof deck panels, a line of panels was omitted over each girder. This 4 ft x 6 in. gap was cast-in-place and contained negative steel (see Fig. 7) which developed continuity in the roof girders for the live load and all dead load above the purlins.

Fig. 5—Column base after grouting; welded column base with 1½-in. anchor bolts; space between column and footing filled with dry-packed concrete

Similarly at the second floor level, the 40-ft girders were shored at midspan prior to the placing of the 5 in. thick floor slab. Added rein-

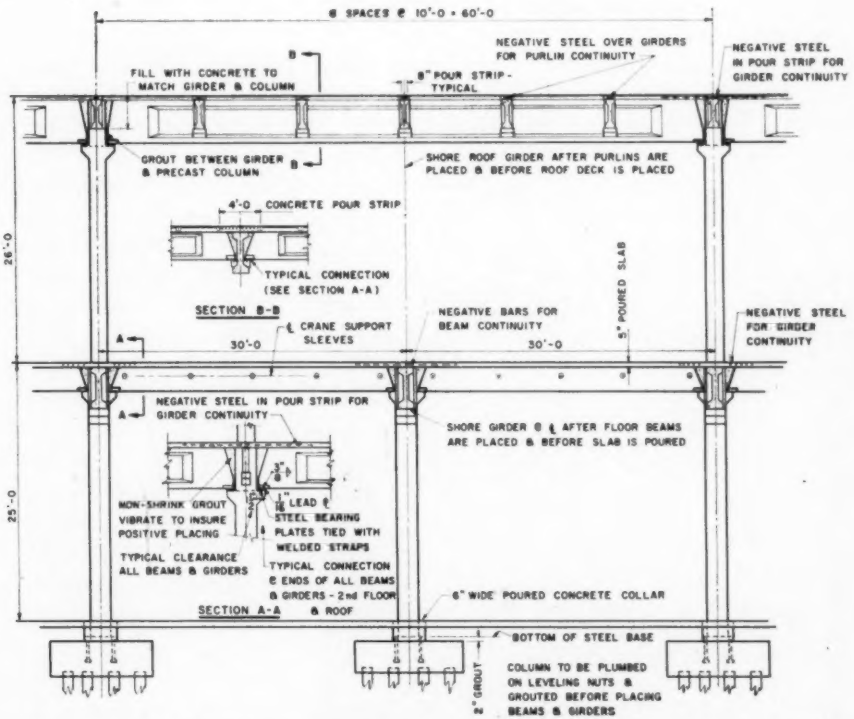
6 SPACES @ 10'-0 = 60'-0
NEGATIVE STEEL OVER GIRDERS FOR PURLIN CONTINUITY
NEGATIVE STEEL IN POUR STRIP FOR GIRDER CONTINUITY
FILL WITH CONCRETE TO MATCH GIRDER & COLUMN
8" POUR STRIP - TYPICAL
GROUT BETWEEN GIRDER & PRECAST COLUMN
SHORE ROOF GIRDER AFTER PURLINS ARE PLACED & BEFORE ROOF DECK IS PLACED
4'-0 CONCRETE POUR STRIP
26'-0
TYPICAL CONNECTION (SEE SECTION A-A)
SECTION B-B
CRANE SUPPORT SLEEVES
30'-0
NEGATIVE BARS FOR BEAM CONTINUITY
30'-0
5" POURED SLAB
NEGATIVE STEEL FOR GIRDER CONTINUITY
NEGATIVE STEEL IN POUR STRIP FOR GIRDER CONTINUITY
SHORE GIRDER & AFTER FLOOR BEAMS ARE PLACED & BEFORE SLAB IS POURED
25'-0
NON-SHRINK GROUT VIBRATE TO INSURE POSITIVE PLACING
TYPICAL CLEARANCE ALL BEAMS & GIRDERS
½" LEAD STEEL BEARING PLATES TIED WITH WELDED STRAPS
TYPICAL CONNECTION @ ENDS OF ALL BEAMS & GIRDERS - 2nd FLOOR & ROOF
SECTION A-A
6" WIDE POURED CONCRETE COLLAR
BOTTOM OF STEEL BASE
2" GROUT
COLUMN TO BE PLUMBED ON LEVELING NUTS & GROUTED BEFORE PLACING BEAMS & GIRDERS

Fig. 6—Typical interior cross section of building showing basic details used to achieve complete continuity in the structure

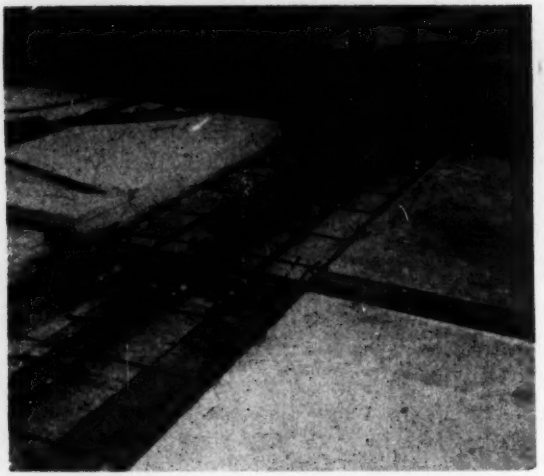

Fig. 7—Overlapping negative steel of both roof girder and roof beams; precast roof panels carried by prestressed purlins; space 4 ft wide over girders is formed for in situ concrete; negative steel over purlins and girders develops continuity in these members

forcement in the second floor slabs at the columns developed continuity in the girders. The shores were struck after the cast-in-place concrete gained sufficient strength, and subsequent loadings were carried by composite action.

For the roof girders and the second floor girders and beams, the additional flange furnished by the cast-in-place concrete was assumed effective in T-beam action to resist compressive stresses. The tops of the precast members were left rough and stirrups extended into the cast-in-place concrete to tie the elements together and develop horizontal shear.

Expansion type grout was used in V-joints between roof deck panels to distribute concentrated loads and develop a diaphragm action in the roof. However, any T-beam action from the participation of the roof deck was ignored in the design of the purlins.

The beams and girders bear on corbels cast on the columns and girders. Two-pound lead sheets were used to distribute possible high

Fig. 8—Welding roof girder to column corbel

bearing concentrations between members. The corbels on the columns and girders had anchored plates or angles which were capable of transmitting tension. Fig. 8 shows a roof girder being welded to a column corbel. Steel sole plates or angles embedded and anchored in the beam and girder ends were welded to the corbel steel to develop a tensile moment connection for possible moment reversals due to lateral forces. The space between member ends and column or girder was filled with nonshrinking grout (by the addition of aluminum powder) and was assumed to carry the compression stresses caused by the continuity of the frame. Corbels were designed for maximum shear although undoubtedly the filled portions share in the vertical shear as well as compression.

DESIGNING FOR FLEXIBILITY IN OCCUPANCY

A developmental center for a company engaged in the development, systems management, design and manufacture of jet aircraft, missiles, and space vehicles obviously required a building which would easily accommodate the many inevitable changes in occupancy. An important design criterion was that any of the space be adaptable for use as laboratories and test areas or for light manufacturing, assembly and mock-ups. This consideration determined the 60 x 40 ft column spacing for the second floor; but it also caused the adoption of many other design details.

To provide flexibility for equipment installation and for supporting piping and conduits, continuous steel channel inserts were provided in the center of the bottom of all purlins, beams and girders and vertically in the four faces of all columns (see Fig. 9). These inserts and their anchors were rated to support loads of 1000 lb per linear ft. This system of inserts saved more than their original cost in the reduced labor of installing the original building services: piping, trays for electrical feeders, electrical conduits, lighting fixtures, and ventilating ducts. More than 27 miles of these inserts were used in this building. See Fig. 10 for

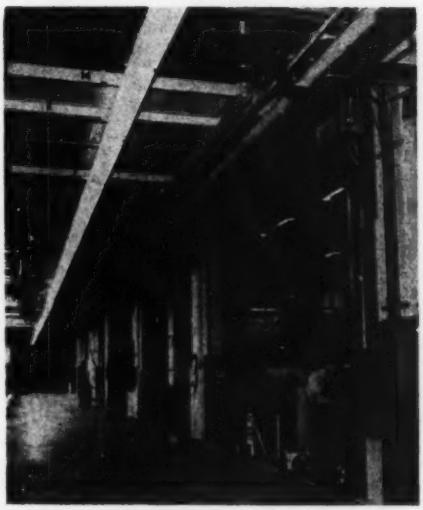

Fig. 9—Continuous steel inserts on the face of precast concrete columns and prestressed concrete beams and girders used in the framing facilitated installation of all types of building equipment, including power distribution and control units, lighting fixtures, piping, and overhead conveyors

Fig. 10 — Typical second floor space showing fixtures, duct work, and other service equipment suspended from inserts in the framing

a typical second floor space showing fixtures, duct work, and other service equipment suspended from inserts in the framing.

To provide for the support of heating and ventilating equipment platforms, cranes, monorails (31 monorail systems were installed initially), and other heavier loads, a pattern of 2 in. diameter pipe sleeves was provided in the webs of all purlins and second floor beams. In the purlins these sleeves were spaced at about 9-ft centers; in the second floor beams at about 6-ft centers.

These web sleeves were also used in construction to support the forms for the second floor slab. Before erecting the second floor beams, timber ledgers were bolted against each side of the rectangular beams by bolts through the web sleeves. The ledgers supported plywood form panels flush with the top of the beams.

To accommodate future service connections through the second floor slab, 2 in. diameter tapered cylindrical wood inserts were set on the forms to reduce the thickness of the slab to 1 in. This provided easily broken out spots, free of reinforcing steel, on a pattern of 5-ft centers in both directions throughout the second floor slab.

ASSEMBLY LINE FABRICATION OF STRUCTURAL FRAME

The structural frame elements are described in Table 1. Because of the large number of second floor beams and their rectangular cross section it was found that they could be produced more economically in a shop with gang forms and then trucked to the job. Fig. 11 shows a 60 ft roof girder moving from the casting bed to the point of erection.

The other 1369 structural members were fabricated on the site by setting up an assembly-line operation on the 13,000 sq ft concrete apron previously mentioned.

The maintained schedule called for all framing elements for two 60 x 40-ft bays (equivalent to 9600 sq ft of floor area) per day. Thus one day's requirement was:

2	50-ft columns	12	40 ft roof purlins
2	25-ft columns	4	40 ft second floor girders
2	60 ft roof girders	16	30 ft second floor beams

Because of the exterior columns and the doubled columns at the expansion joints, the production schedule for columns was doubled and four of each length were produced per day.

TABLE I — PRECAST STRUCTURAL FRAME DETAILS

Pieces	Member	Weight, tons per member
153	50-ft columns	14
131	25-ft columns	8½
132	60 ft roof girders	16
709	40 ft roof purlins	5
244	40 ft second floor girders	11½
920	30 ft second floor beams	3½
2289 total pieces		total weight = 15,000 tons

For all precast members at the job site the concrete was required to reach a strength of 4500 psi in 3 days. This was accomplished by using a 7.2 bag mix of Type III cement, a 1 in. maximum slump and both external and internal vibration.

Concrete was placed in the forms with a portable conveyor (Fig. 12) which was attached to the rear of a truck mixer and followed it down the center roadway parallel to the forms. By this method the 8 cu yd of concrete for a 60-ft girder could be placed in about 10 min.

Wood forms with a fiberboard lining were used for all columns. Special precautions were observed in accurately positioning the steel angle base frame and the bearing plates on the corbels. The 50-ft columns were lifted at two points 3 days after they were cast.

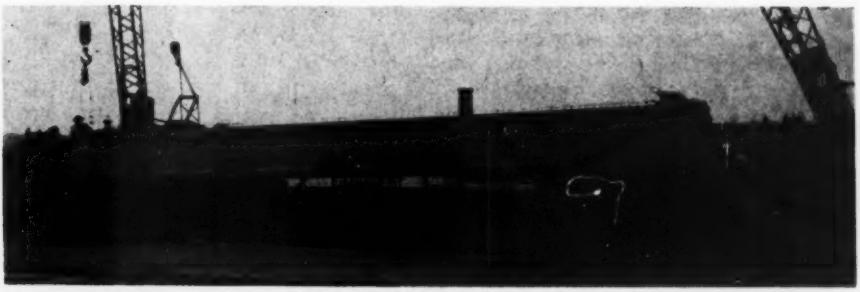

Fig. 11—60 ft roof girder (weight 16¼ tons) placed on tractor unit and separately steered trailer

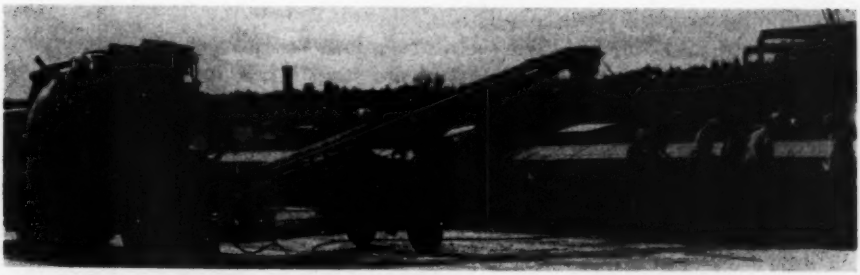

Fig. 12—Placing concrete into roof beams using electric conveyor belt; high strength concrete is belt-conveyed from mixer truck to girders; externally-vibrated steel forms compact the 1-in. slump concrete

Fig. 13—Forms for second floor girder corbels installed ready for concrete; corbel forms were bolted to prestressed girders, simplifying the forming and casting of the girders

Steel forms specially fabricated from 3/16-in. plate were used for the second floor girders and the roof girders and purlins. The number of side forms was equal to the number of members scheduled per day; for example, there were two forms for 60 ft roof girders. There were four times as many bed plates as there were sets of side forms, as each member remained on its casting bed 3 days before being post-tensioned and moved.

It was found advantageous to strip side forms as soon as 4 hr after casting to avoid cracking due to differential shrinkage of the concrete and temperature movement of the steel forms. The stripped forms were then cleaned and set on the next bed plate.

Reinforcing steel and the prestressing tendons were completely preassembled, and could therefore be placed in the forms with a crane in a matter of minutes. The continuous insert required in the bottom flange of all members was held on the bed plate by a snap fastener that both permitted its easy placement and removal of the finished beam.

Column corbels were cast integrally with the columns. The corbels on the sides of the girders were cast after the I-section forms were removed. Fig. 13 shows a second floor girder corbel form installed and ready for concrete. This procedure simplified the forming and stripping of the girder side forms and side-stepped the potential problem of shrinkage cracking the projecting corbels had they been cast monolithic with the girder. The reinforcement of the corbels was placed through sleeves in the girder webs, and steel forms for the corbels were clamped to the girders. Load tests conducted on corbels cast against the girder in this manner indicated no loss in shearing strength because of the cold joint.

The Freyssinet system was used for the prestressing of girders and beams: the cables were tensioned with portable double-acting jacks which also seated conical wedges to anchor the individual wires (see Fig. 14).

All tendons consisted of twelve 7-mm wires with a minimum tensile strength of 236,000 psi. Although the wire was stress relieved it was

found desirable to straighten it to speed its handling. A job-built straightening machine solved this problem satisfactorily.

After straightening, the wire was cut to length and assembled in flexible steel tubes. It was then placed in the prefabricated reinforcing cages where the proper location of straight and draped tendons was insured by a series of short pencil rods welded to the stirrups. Fig. 15 shows the installing of a reinforcing cage for a roof girder.

Fig. 14 — Post-tensioning roof beams with Freyssinet jack; portable safety shield mounted on casters is provided with boom and chain hoist for moving the jack

The 60 ft roof girders and the 40 ft second floor girders had two straight tendons and either two or three draped tendons depending on the member's location in the frame. All 40 ft roof beams had only two draped tendons. After tensioning all tendons were filled with a special grout made up of heat cement, 5 gal of water, 2 ounces of retarding densifier, and 2 g of unpolished aluminum powder.

The second floor beams were cast with 6000-psi concrete and were a combined pretensioned and post-tensioned design. The straight tendons were pretensioned ⅜-in. diameter, seven wire strands with a minimum tensile strength of 250,000 psi. One draped tendon containing twelve 7-mm wires was post-tensioned.

ERECTION OF THE STRUCTURAL FRAME

The general erection procedure was essentially similar to that used for a structural steel frame.

Each column footing had four 1½ in. diameter anchor bolts on which a leveling nut was first placed. The column was then placed on the leveling nuts which were adjusted to set the column to grade, and plumb. Two hand jacks assisted in this operation. A second nut was screwed down to secure the column to the footing. The space between the column base and the pile cap was dry-packed with nonshrinking grout containing aluminum powder to prevent shrinkage.

After setting the columns, the roof girders and purlins were erected, and their sole plates welded to the bearing plates. Then the second floor

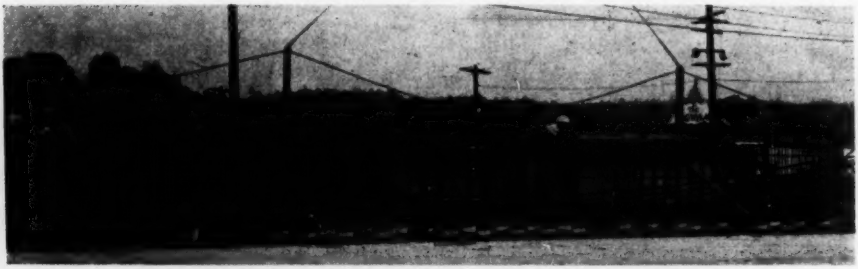

Fig. 15—Installing reinforcing cage for a roof girder

girders and beams were erected and their welded connections made. An end clearance of 1½ in. on the girders, purlins and beams made the erection quite simple, and no difficulty was experienced in placing any of the members.

The more than 6000 precast roof panels were placed on the purlins with portable equipment. These specially designed 4 x 10-ft units, cast of lightweight concrete, weighed 800 lb and were handled with ease using a rubber-tired dolly.

The precast roof panels and the use of second floor form panels supported by the second floor beams eliminated the extensive shoring which would have been required because of the 25 ft story heights. The only story-to-story shores were the previously mentioned temporary midspan supports under the roof and floor girders to secure integral action of these girders with the slabs above them in carrying a part of the dead load as well as the live load.

WALL PANEL DESIGN

One of the unusual features of this building is the extensive use of large precast, prestressed wall panels for the exterior walls. These panels were 5 in. thick and were designed to span from column to column: 30 ft on the west side of the building and 40 ft on the north and south sides. On the 40-ft spans intermediate lateral support is supplied by a steel wind column.

The 70 precast wall panels provided 44,000 sq ft of wall surface. The largest panel was 41 x 20 ft. This type of panel was used not only because it proved most economical for the criteria, but also because it would permit easy removal and re-use in the event of future additions to the building. The panels were prestressed to prevent cracking from handling during erection and from temperature strains after erection, and also to eliminate the possibility of warping during curing. They were pretensioned longitudinally with 7/16 in. seven-wire strands to produce a uniform initial compressive stress of about 500 psi in the concrete. The spacing of the strands varied in the different size panels

but approximated 7 in. Transverse reinforcement was mild steel. See Fig. 16 for a general view of wall panel strands and reinforcing.

The aggregate was Haydite giving a concrete density of about 105 lb per cut ft. The minimum 7-day strength of this concrete was 4000 psi.

The entire weight of the panels, at erection, was taken by heavy steel shelf angles on the columns. Lead pads were provided to distribute possible high concentrations of load.

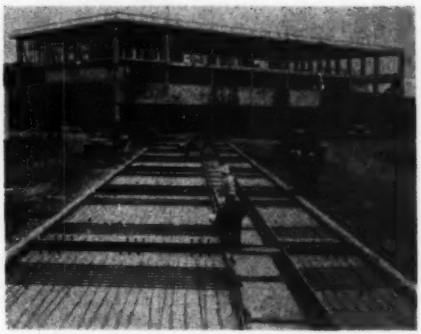

Fig. 16—General view of wall panel strands and reinforcing

Lateral support for the walls was provided by a vertical row of clip angles at each column and also by vertically slotted clip angles at the second floor line. The clip angles on the columns were designed to have a limited freedom of horizontal rotation on the column but were rigidly bolted to the wall panel and are capable of ultimately sharing in vertical load. This flexible arrangement allows the wall panels to expand and contract with temperature changes; and because of prestressing the panels are prevented from cracking. The ½-in. joints between abutting panels were filled with a permanently resilient caulking compound.

FABRICATION OF PRETENSIONED WALL PANELS

An economical wall-panel prestressing bed was secured by using one of the pavement slabs cast on grade. This 8 in. plain concrete slab was analyzed for prestressing loads and found to be adequate with only a nominal amount of thickening and reinforcing at the ends. The slab was cast with a 1 in. slump concrete, monolithic for the entire length of 260 ft. No shrinkage cracks developed and it proved to be an excellent pretensioning bed. For the 20 ft high panels, a total pretensioning force of 600 kips was carried by thirty-two 7-wire strands. These strands were stressed in groups of eight and anchored by individual grips. A plan of the casting bed for pretensioning wall panels is shown in Fig. 17.

The lightweight concrete was leveled with a rotating screed, and coarse-broomed for texture. A 7.4-sack, Type III cement mix reached a compression strength of 4000 psi in 4 to 5 days.

To permit the simultaneous and controlled release of the pretensioning load on as many as 32 strands over the 20 ft wide bed, eight horizontal cylindrical paraffin rams were used between a fixed and a sliding end

block (see Fig. 18 and 19). These specially designed rams were 5 in. in diameter and carried 75 kips each.

Before the strands were tensioned (at the opposite end of the bed) the rams were filled with paraffin and properly positioned. At normal temperature the paraffin was solid.

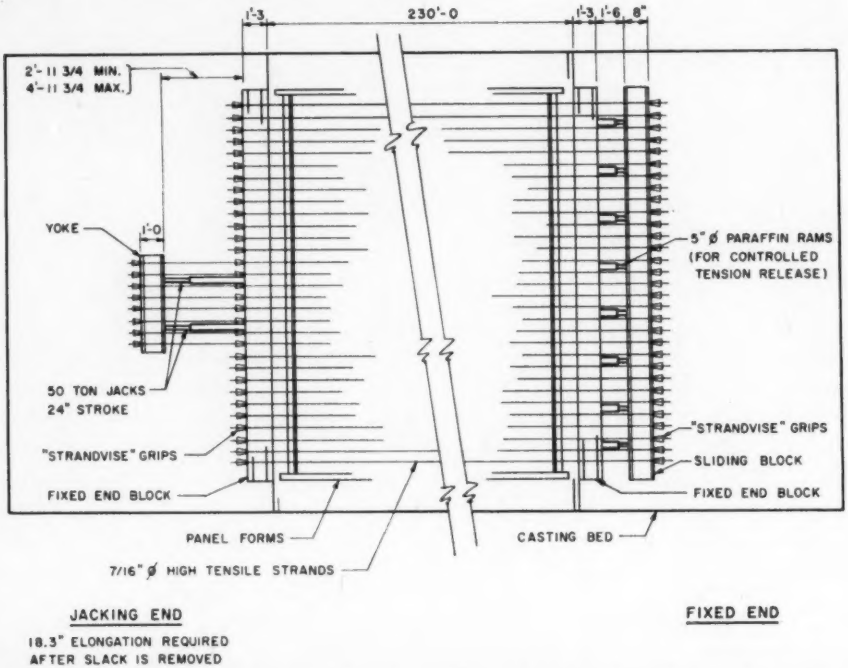

Fig. 17—Plan of concrete casting bed for pretensioned wall panels showing jacking arrangement and paraffin rams for controlled release of tension in strands

Fig. 18—Paraffin jacks being heated; emitting liquid paraffin; transferring stress on wire to the wall panels

Fig. 19 — Stressing strands showing shield, jacks, and pumps

To release the tension on the strands, a bleed valve on each cylinder was opened and the cylinders were heated until the paraffin softened and extruded through the valves. The rate of flow was controlled both by the valves and by the electrical heat. This system performed successfully and afforded complete control over the release of tension. After release of tension, the strands were cut flush with the ends of each wall panel. The panels were then picked up with a crane and stored in a rack vertically until ready for erection.

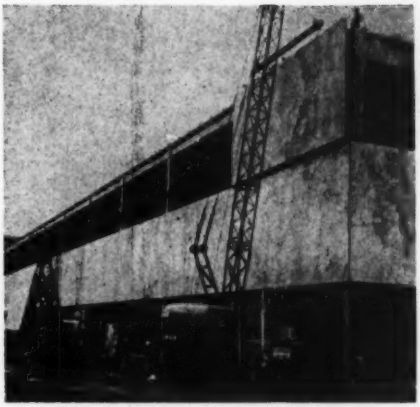

Fig. 20—North wall of Building 9-101; upper panel being installed; panel size 41 x 17 ft

WALL PANEL ERECTION

Two loops made of ⅝ in. diameter 7-wire strand were embedded in the top edges of each wall panel. A wire rope bridle was hooked into these loops, and the crane hook engaged the bridle to lift the panel from the trailer. The panel was held in place against the column until seated and bolted to the clip angles on the column. See Fig. 20 for a view of an upper panel wall being installed; visible beyond the crane is the special trailer capable of transporting two 20-ton panels back-to-back from storage area to the building.

CONCLUDING REMARKS

The erection of the thousands of precast and prestressed concrete members was carried out without serious accident or damage to the structural elements, and as scheduled. By allowing liberal tolerances in the over-all dimensions of the members whenever possible, no com-

Fig. 21—South elevation at east-west expansion joint with roof panels partly in place

Fig. 22—Underside of roof slab at expansion joint; opening in roof for air intake.

plication in fitting the parts together was experienced.

The concept of joining pieces of prefabricated concrete together, with the object of achieving the continuity of a monolithic structure was successfully demonstrated in this project.

This structure has been completed and been in use for over 3 years, which should be sufficient time for any design or construction defects to become apparent. A thorough inspection of the building was made in January, 1961. This inspection revealed no cracks in any of the prestressed members and the prestressed exterior wall panels. No movement or cracking was observed at connections between beams, girders, and columns.

From paint markings along the east-west expansion joint at the second floor it is evident that this joint had closed ¼ in. more than its position in January (i.e., the joint is working as planned). No such indication of movement was observed at the north-south expansion joint. Fig. 21 and 22 show two instances of an expansion joint.

In reporting the cost of this building it is necessary to note (see Fig. 1) that the east end of the concrete structure abuts a steel-framed structure, and at the southwest corner there also is included another steel-framed structure. Both of these steel-framed structures are at least as high as the concrete building and their exterior walls are insulated steel siding. The extent of exterior precast concrete wall panels is therefore less than if the concrete building were free standing. Therefore in compiling the actual cost of this structure, the cost of additional precast wall panels was added to give a completely enclosed building; with this addition, the actual construction cost (1957) was $4.90 per sq ft of floor.

This cost includes the foundations, the concrete floor on grade, all concrete framing both precast and cast-in-place, the cast-in-place second floor slab, a full exterior wall of prestressed panels, the precast roof slabs and all cast-in-place strips, the roof waterproofing, and the contractor's profit. The concrete footings as installed are adequate for normal bearing soils, so the cost of the timber piling has been excluded to give a more generally applicable cost figure.

ORGANIZATION

The designers of the Developmental Center were Austin Associates with A. T. Waidelich in responsible charge. Anderson, Birkeland and Anderson were consultants on the prestressed design and construction of this project with Arthur Anderson in charge. Construction of the entire developmental center was by The Austin Co., with W. R. Engstrom, vice-president, in charge.

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Concreto prefatigado y premoldeado La edificación en la instalació Boeing

Se describen las características particulares del diseño y de la construcción de un edificio de dos pisos, construído de concreto premoldeado y prefatigado, el cual se encuentra en el Centro de desarrolo de la Boeing Airplane Cia., Seattle, Washington. El área del piso tiene más de 500.000 pies cuadrados, el cual fué moldeado, erigido y completado en unos 14 meses. Los pisos tienen cada uno 25 pies de alto. El espaciamiento de las columnas es de 60 x 40 pies en el segundo piso y de 30 x 40 pies en el primer piso, de modo que las longitudes de las columnas son alternadamente de 50 y 25 pies.

Los detalles y los procedimientos de construcción fueron empleados para poder conseguir continuidad completa en todas las juntas de los miembros premoldeados y se realizó también la participación de la cubierta del techo y de la losa del segundo piso en el soporte de la carga.

La mayor parte de los miembros premoldeados y prefatigados fueron fabricados en el sitio, inclusive los tableros de pared de concreto liviano soportados en el extremo, los cuales tiene un máximo de 20 pies de alto y un tramo máximo de 40 pies.

Bâtiment en béton coulé d'avance et précontraint, â l'usine Boeing

Décrit les advantages saillants de la conception et construction d'un bâtiment à deux étages en béton coulé d'avance et précontraint. au Developmental Center de la Boeing Airplane Co., à Seattle, Washington. La surface du plancher, plus que 500,000 pieds carrés, était coulée, installée et finie en 14 mois à peu près. L'hauteur des étages est de 25 pieds chacun. L'èspacement des colonnes est de 60 x 40 pieds au 2e étage et 30 x 40 pieds au rez de chaussée, donc les colonnes ont, alternativement, 50 et 25 pieds de longueur.

On utilisait des détails et des procédés de construction produisant une continuité parfaite à tous les joints des organes coulés d'avance, et la participation au support de la charge du faux coble et des dalles du 2e étage.

La plupart des organes coulés d'avance et précontraints étaient fabriqués in situ, ci-inclus les panneaux de parois en béton léger précontraint, supportés aux bouts et jusqu'à 20 pieds de hauteur, ayant une portée maximum de 40 pieds.

Gebaeude aus vorgespanntem vorgegossenen Beton in Boeing Werk

Beschreibt die ausgepraegten Konstruktions- und Baumerkmale eines zweistoeckigen Gebaeudes aus vorgespanntem und vorgegossenem Beton in Entwicklungsinstitut der Boeing Airplane Co. in Seattle, Washington. Die Bodenflaeche war mehr als 500,000 Quadratfuss, die in ungefaehr 14 Monaten gegossen, errichtet und fertiggestellt wurde. Jedes Stockwerk ist 25 Fuss hoch. Der Saeulenabstand ist 60 x 40 Fuss auf dem zweiten Stock und 30 x 40 Fuss auf dem ersten Stock, sodass die Saeulenlaengen abwechselnd 50 und 25 Fuss sind.

Die verwendeten Einzelheiten und Baumethoden ergaben einen vollstaendigen Uebergang bei allen Verbindungsstellen der vorgegossenen Glieder und Beteiligung des Daches und der Betonplatte im zweiten Stock an der Traglast.

Die Mehrzahl der vorgegossenen und vorgespannten Glieder wurden an Ort und Stelle hergestellt, einschliesslich der entgelagerten, vorgespannten Leichtbeton-Wandplatten von 20 Fuss Hoehe und einer Maximal Spannweite von 40 Fuss.

Title No. 58-3

Properties of an Expansive Cement for Chemical Prestressing

By ALEXANDER KLEIN, TSEVI KARBY, and MILOS POLIVKA

Expansion characteristics and compressive strengths were determined for concretes containing expansive cement to evaluate the factors relating to control of the expansive reaction. The expansive cement consists of a portland cement component and a calcium-sulfoaluminate anhydrite component. The factors influencing the magnitude and rate of the expansive reaction include: chemical composition of the components, fineness of the sulfoaluminate component, proportions of the two components in the total cementing material, ratio of water to total cementing material, richness of mix, conditions of curing, and degree of restraint.

In this investigation, the chemical composition of the components and the fineness of the sulfoaluminate component were kept essentially constant, and other factors influencing magnitude and rate of expansion were varied.

For this study, restraint in all cases was provided by external steel mechanisms, in some cases restraint being uniaxial, and in other cases biaxial. In the absence of restraint, concretes exhibited free expansions up to 6 percent or more.

The properties of the concretes reported herein indicate that certain of the expansive cements tested are suitable for structural work and can effectively be employed in the manufacture of chemically prestressed members under conditions of external restraint. It was established that with proper mix proportioning and curing, with compositions, proportions, and fineness of components fixed, it is possible to produce concretes having desired predetermined characteristics within a practical range.

■ ABOUT 70 YEARS AGO Candlot[1] established that the interaction product of tricalcium aluminate and calcium sulfate in an aqueous medium produced a hydrated salt of calcium sulfoaluminate, long known as Candlot's salt (now known as ettringite). Michaelis[2] proposed that this reaction, when occurring between the tricalcium aluminate in portland cement and sulfates from aggressive waters, was responsible for the expansive disruption of otherwise sound concrete; hence, he called this salt "cement bacillus." Nearly 40 years later, Lerch, Ashton, and Bogue[3] concluded that there are in fact two calcium sulfoaluminate hydrates, the low monosulfate (of molecular composition $3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 12H_2O$) and the high trisulfate (of molecular composition $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 30\text{-}32H_2O$).

The low sulfate apparently does not contribute an expansive reaction on formation, whereas ettringite forms with substantial expansion, and

under restraint exerts a large expansive force against such restraint. According to Ferrari,[4] a controlled formation of expansive sulfoaluminate will occur only in a medium supersaturated with respect to calcium ions, and maintained at supersaturation during the formation of this salt.

Following the work of Candlot and Michaelis, chemists, engineers, and researchers sought diligently to find means of exploiting the considerable expansive forces set up during the formation of ettringite. The Lafarge Co. found early that the reaction obtained using a mixture of gypsum or plaster, aluminous cement, and portland cement was both erratic and difficult to control. Subsequently, Lossier[5] in France used an expansive clinker claimed to be calcium sulfoaluminate. His method of control consisted in regulating availability of water to the concrete and insuring termination of reactions by including blast-furnace slag in the mixture. However, Lafuma,[6] also in France, reported that the clinker consisted of essentially dicalcium silicate, calcium sulfate, and various calcium aluminates and ferrites.

More recently, considerable research has been carried on by Mikhailov[7,8] in the USSR with self-stressing cement "SC," also consisting of a mixture of portland cement, gypsum plaster, and aluminous cement. Hydrothermal methods of curing, which are closely controllable, are used to achieve control of the expansive self-stressing reactions.

Most recently, a component for expansive self-stressing cement has been developed at the University of California,[9] consisting of a clinker proved by x-ray diffraction and petrographic examinations to be, for the most part, a true anhydrous calcium sulfoaluminate, the remainder being free lime in an amount that may be between 15 and 20 percent of the composition.

Factors and mechanisms influencing rate and magnitude of expansion

For practical use of the expansive energy generated in the formation of ettringite, it is required to control the magnitude of expansion, the rate of expansion, and the conditions of restraint. The magnitude desired may be that contemplated for producing a so-called nonshrinking cement, or again for producing a "stressing" cement. Nonshrinking cements are, in fact, compensating cements. They are designed to produce relatively small expansions in concrete, of magnitude sufficient to compensate for drying shrinkage expected at some predetermined age of concrete. To prevent cracking of concrete, at least sufficient restraint is required to impose a degree of compression in the concrete, at early ages, adequate to reduce tensile stresses set up through restrained drying shrinkage to values less than the tensile strength of the concrete at any age. For self-stressing cements, expansions in concretes under adequate restraint must be relatively high, and of magnitudes sufficient to develop economical self-stressing of steel and of concrete, to desired levels over and above stress losses due to creep and to drying shrinkage.

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Tsevi Karby was a graduate student in civil engineering, University of California, at the time of this investigation and used a part of this research in partial fulfillment for his MS degree.

ACI member **Milos Polivka** is an associate professor of civil engineering, University of California. He has been on the faculty since 1948 and has been associated with various research programs in the field of cement and concrete. He is a member of ACI Committee 609, Consolidation of Concrete, and currently is the president of the ACI Northern California Chapter.

The great potential of chemically prestressed concretes can be realized industrially, but only if positive and adequate control is achieved both as to the magnitude and the rate of expansive reaction under available conditions of restraint. It is now known that factors influencing the control of the reactions involve the compositions of the two components, portland cement and expansive clinker, the fineness of the ground expansive clinker, and the relative proportions of the two components. With these factors held constant and uniform, control of self-stressing involves only the factors of richness of mix, ratios of water to cementing material, conditions of curing, and conditions of restraint.

Chemical Mechanisms

The expansive clinker produced at the University of California, having the compositions reported by Klein and Troxell,[9] or superior compositions developed since, consist for the most part of anhydrous calcium sulfoaluminate of as yet undetermined molecular composition, and substantial amounts of free lime. The presence of free lime is apparently essential to the initial formation of ettringite and insures that solutions remain supersaturated with respect to calcium ions during the active stages of formation and attendant expansion. It is noteworthy, as previously reported,[9] that little or no calcium hydroxide is found even at early ages of hydrates of the expansive component with free lime contents as high as 20 percent. Although the amount of free lime in the sulfoaluminate anhydride may be high, the concentration of calcium ions in the liquid phase of lean mixes may be sufficiently low that expansion will not occur with expansive cements containing less than about 10 percent of the expansive component.

The necessity of supersaturated solutions with respect to lime is indicative of the desirability of low ratios of water to cementing material in increasing the expansive potentials when using the expansive cement in concretes at a given richness of mix. The lower the water content of the plastic paste, the greater the concentration of calcium ions in the liquid phase. The limited data presently available indicate strongly that to best insure appropriate conditions for the formation of the ettringite

the portland cement component of the expansive cement preferably should be high in C_3S and low in C_3A.

Factors of mixing, placing, and curing

Methods of mixing, placing, and curing of concretes containing expansive cement of the type prepared at the University of California are entirely conventional. The concretes are rather fast setting with low ratios of water to cementing material. With high percentages of expansive component and rich mixes, a water-reducing retarder can be effectively employed to improve workability and further decrease the ratio of water to cementing material without adverse effects on expansive forces. The degree of free expansion is controlled through the use of expansive cements of fixed composition and uniform quality, and through the selection of appropriate richness of mix and water-cement (W/C) ratio.

The methods of curing these expansive concretes also are conventional, but can be modified in such a manner as to control the rate of expansion. While curing in water produces the most rapid expansion, nearly the same rate of expansion is produced with fog curing at the same temperature. Unlike the Lossier cements, or the USSR "SC" cements, expansion proceeds even in specimens sealed immediately after casting, although at slow rates and at reduced magnitudes at final equilibrium. A particularly effective method of controlling the rate of expansion in concretes where high expansions are desired is merely to cover the concrete, after stripping the mold, with a loose-fitting membrane in fog for about 1 day, and subsequently to cure in fog or water. Thus time is allowed for development of adequate strengths to resist self-destruction of the concrete through stresses imposed by restraint under conditions of high and rapid expansion.

Factors of restraint

With unrestrained expansion, the forces created are disruptive and concrete strengths are diminished proportionally to increases in free expansion. If complete restraints are provided such that expansion cannot occur, compressive stresses will be high, but there can be no self-stress prestress in concrete on removal of restraint. The magnitude of expansion which takes place under restraint will govern the tensile strains in restraining steel, and depending on the relative cross sections of steel and concrete, the compressive self-stress in the concrete as long as restraint is maintained. It is of course required that anchorage of steel in the concrete be adequate to prevent slip of concrete with respect to steel at high stress values in steel. The higher the expansion under restraint, other things being equal, the higher will be the self-stresses in steel and in concrete, even to the point of self-destruction if the rate of increase in self-stress is more rapid than the rate of increase in the

compressive strength of the concrete. It is to be expected that under conditions where the stress-strength ratio approaches unity throughout much of the period of major expansion, creep will be high. However, it can be expected that on completion of the expansion stage, the concrete will be substantially free of stress-loss effects due to creep under subsequent conditions of drying shrinkage.

Other than the aspects of control of the expansive reaction, there is an inherent and as yet unexplained characteristic of expansive concretes of great significance in future applications. Although precise determinations have not yet been made at the University of California, there is considerable evidence that in small specimens with symmetrical restraint the expansive movements in all directions are substantially equal per unit length of concrete even under uniaxial restraint, where the whole cross section normal to the axis of restraint is under adequate compression. This characteristic is cited repeatedly in Russian literature on expansive cements, although without pertinent supporting data. The ability to achieve three-dimensional restraint of concrete with uniaxial steel restraint could open realms of application not possible with mechanical prestressing, even though the expansion may not be equidimensional in relatively large specimens.

CURRENT INVESTIGATION

The first stage of the investigation carried out at the University of California, as reported by Klein and Troxell,[9] involved a study of various calcium sulfoaluminate compositions. As a result of this study a calcium sulfoaluminate clinker was produced in sufficient quantity to carry out an extensive investigation on concretes containing this expansive component. Concretes containing various amounts of expansive component were investigated under either external biaxial or external uniaxial restraint to develop chemical prestress. In chemical prestress the energy of the expanding concrete, during hardening, is used to develop compressive stresses in concrete and tensile stresses in restraining steel. This chemical expansion tensioning differs from the mechanical pretensioning or post-tensioning systems in that the prestressing is developed gradually during the curing of the concrete.

Cylindrical concrete specimens were employed for the study of biaxially restrained concretes, and rectangular concrete bars were used for the uniaxially restrained concretes. Also tested were concrete pipes under external biaxial restraint. Because of the difference in the type of specimen employed for each of the two types of restraint, separate determinations were required for the free-expansion characteristic and for the compressive strength of unrestrained concretes.

Factors evaluated included the effects of curing conditions, amount of expansive component, richness of mix, W/C ratio, and use of a water-

reducing retarder. The main effort was focused on the development of self-stress and the gain of compressive strength.

All tests herein reported were performed at a temperature of 70 F. The amounts of expansive component and the ratios of water to cementing material are expressed herein always by weight. The amount of expansive component is expressed as the percentage of the total cementing material, and the richness of mixes is always expressed in terms of the number of 94-lb sacks of cementing material per cu yd of concrete.

The chemical compound compositions of the expansive components and the Type I portland cement employed in this investigation were described by Klein and Troxell.[9] The expansive component used in concretes for the uniaxially restrained specimens were of compositions superior to those previously reported.

TEST RESULTS OF SERIES I—CONCRETES UNDER BIAXIAL RESTRAINT

In this phase of the program a study was made of the effect of curing and of the amount of expansive component on the compressive strength and on the expansive characteristics of concretes subjected to biaxial restraint. The expansive characteristics were determined for externally restrained concrete specimens. Compressive strengths were obtained both for unrestrained and for biaxially-restrained specimens. The concrete mix employed had a cementing-material content of 8.0 sacks (752 lb) per cu yd and a W/C of 0.45 by weight, and contained ¾ in. maximum size aggregate. The nominal slump was 2 to 3 in. All concretes were mixed in a laboratory-size pan-type mixer.

The amounts of expansive components investigated ranged from 12.5 to 25 percent by weight of cementing material. The cementing material containing Type I portland cement and the expansive component was blended in a rotary blender. In practice the expansive component could be interground at the cement plant to insure required uniformity and close control of proportions.

The concrete specimens employed in this phase of the investigation were 3 x 6 in. cylindrical specimens, provided with stainless steel gage plugs at their ends. A dial gage extensometer was used to measure the free expansion. Specimens of the same dimensions were employed in compressive-strength tests.

Biaxial restraint of cylindrical concrete specimens was accomplished by leaving the specimens in the heavy cast-iron molds in which they were cast. The 3 x 6 in. molds provided restraint only along the sides of the specimens; the ends were unrestrained. Longitudinal restraint was not as positive as circumferential restraint; it was by friction only. To discontinue restraint, molds were removed from the specimens.

Unrestrained control concretes

Effect of curing conditions — The cementing material employed in the study of the effect of curing conditions contained 25 percent expansive component and 75 percent portland cement by weight. One group of concrete specimens was cured in fog and the other under drying conditions at 50 percent relative humidity. Results of these tests up to 35 days are shown in Fig. 1. The fog-cured specimens expanded rapidly and developed an expansion well in excess of 6 percent. These specimens exhibited extensive map cracking, but did not fall apart. The high expansion of the unrestrained specimens adversely affected compressive strength; the 28 day compressive strength was only 300 psi. Companion specimens cured under drying conditions expanded 1.1 percent during the initial expansion period, terminating at 4 days. For these specimens, the compressive strength at 1 day was 1150 psi and at 3 days, 1650 psi.

Effect of amount of expansive component — The effect of amount of expansive component on expansion characteristics and on compressive strengths was determined for four groups of unrestrained concrete specimens cured in fog. Results of these tests are shown in Fig. 2.

The first group of concretes used a control mix containing no addition. This concrete had a 28 day compressive strength of 4220 psi.

The use of 12.5 percent of expansive component caused the concrete to expand 0.49 percent within the first 7 days, or about 90 percent of the ultimate free expansion. The 28 day compressive strength was 3350 psi, or about 75 percent of the compressive strength of the concrete without expansive component.

Concretes containing 18.5 percent of expansive component expanded 2.2 percent during the first 7 days, or about 95 percent of the ultimate free expansion. The high expansion of this concrete was related to the low compressive strength (750 psi at 28 days).

The use of 25.0 percent of expansive component caused an exces-

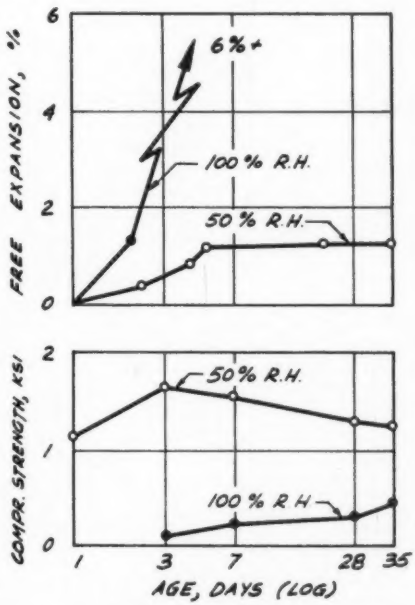

Fig. 1 — Effect of curing conditions on free expansion and compressive strength of concrete containing expansive component. W /C = 0.45 by weight; cement content = 8.0 sacks per cu yd; 25 percent expansive component; curing at 70 F

sively high expansion (above 6 percent) and a low compressive strength (350 psi at 28 days). The deleterious effects of this high expansion were accompanied by extensive map cracking on the surfaces of the concrete specimens. No cracks were evident on concrete specimens containing only 12.5 percent of the expansive component.

These data clearly show that expansion increases with increase in the amount of expansive component, but not proportionally. The expansive reaction was complete at about 10 days.

Stress-strain characteristics — The stress-strain characteristics of unrestrained, fog-cured concretes containing 12.5 percent of expansive component are shown in Fig. 3. For 3 and 7 days there was a well-defined yield point, with considerable yielding at the failure load. As the concrete gained strength with age, the yielding became less pronounced and the concrete failed rather suddenly.

For concrete 3 days old, a yielding of about 0.03 percent was observed at the failure stress of 2200 psi. The secant modulus of elasticity (at 1500 psi) was 1.3×10^6 psi. Concrete specimens 3 months old failed rather suddenly at a compressive stress of 4400 psi; the modulus of elasticity was about 5.0×10^6 psi.

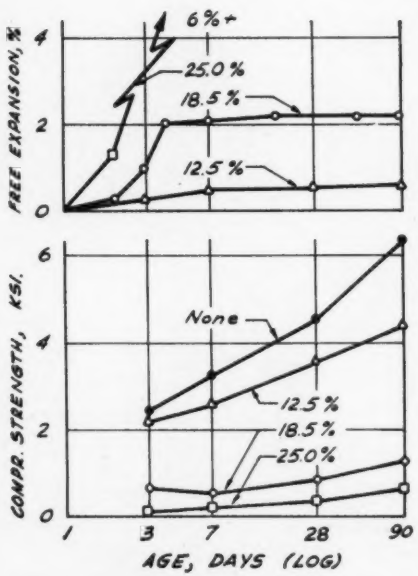

Fig. 2 — Effect of amount of expansive component on free expansion and compressive strength. Amount of expansive component, percent by weight of total cementing material. $W/C = 0.45$ by weight; cement content = 8.0 sacks per cu yd; curing at 100 percent relative humidity and 70 F

The yielding at early ages is typical of concrete containing the expansive addition, regardless of curing or restraint condition. This early-age yielding is indicative of the high creep which must occur in all concretes during the expansion stage if they are restrained throughout this stage. At later ages such concretes have generally the same stress-strain characteristics as concretes of the same age containing no expansive component.

Strength-expansion relationship —The compressive strength of unrestrained concretes containing an expansive component appears to be related primarily to the expansion rather than to the amount of expansive component used. Fig. 4 demonstrates this relationship for concretes 7 days old. Although three of the concretes were fog-cured and one was cured at 50 percent relative humidity, the re-

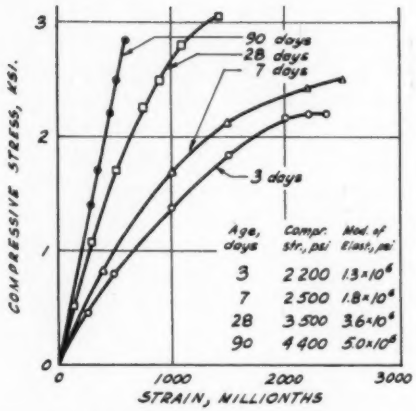

Fig. 3 — Stress-strain characteristics of fog cured unrestrained concrete specimens. $W/C = 0.45$ by weight; cement content = 8.0 sacks per cu yd; 12.5 percent expansive component; curing at 100 percent relative humidity and 70 F

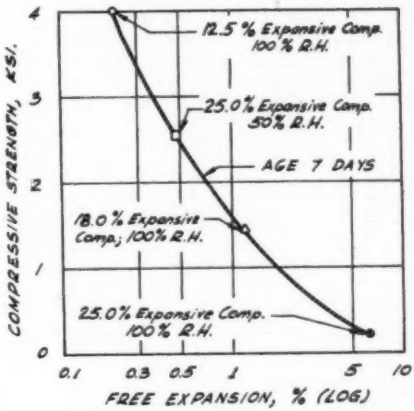
COMPRESSIVE STRENGTH, KSI.
FREE EXPANSION, % (LOG)
12.5 % Expansive Comp. 100 % R.H.
25.0 % Expansive Comp. 50 % R.H.
AGE 7 DAYS
18.0 % Expansive Comp.; 100% R.H.
25.0% Expansive Comp. 100 % R.H.

Fig. 4 — Effect of free expansion on compressive strengths of concretes containing expansive component. $W/C = 0.45$ by weight; cement content = 8.0 sacks per cu yd; curing at 70 F

sults indicate that the compressive strength is closely related to the magnitude of expansion and appears to be slightly affected either by the curing conditions or by the amount of the expansive component, provided the cement factor and W/C are kept constant.

Expansion and strength of biaxially restrained concretes

A group of 3 × 6-in. concrete specimens made with cement containing 25 percent of expansive component was subjected to biaxial restraint during curing. A set of companion specimens cured without restraint was also tested for comparison. All specimens were stored continuously in fog.

The results of the effects of biaxial restraint on expansion and on compressive strength are shown in Fig. 5. Up to 7 days, restrained concretes expanded 0.16 percent, but after removal of restraint a second-stage additional expansion of 0.5 percent was obtained between 7 and 28 days. The compressive strength increased at a uniform rate during the first 7 days, but it decreased rapidly during the rapid second-stage expansion after removal of restraint. This decrease in strength slowed down, as did the expansion, after about 20 days. It should be noted that the external restraining mechanism inhibited absorption of water, whereas internal restraints would not.

The expansion for the companion group of unrestrained specimens (Fig. 5) was high. Most of the expansion took place at an early age when the concrete was weak and could deform plastically. The expansion was essentially completed at 7 days. The low compressive strength is primarily due to the excessively high expansion.

TABLE 1 — MODULI OF ELASTICITY AND RUPTURE

Age	Compressive strength, psi	Modulus of elasticity, psi	Modulus of rupture, psi
40 days	5400	5.0×10^6	1080
1 year	8230	7.6×10^6	1060

Effect of curing conditions—It was shown in Fig. 5 that, for specimens cured in fog, the expansion continued to increase after restraint was removed at 7 days, while the strength decreased due to excessively high expansion. However, when these specimens were stored under dry conditions (50 percent relative humidity), the expansion essentially terminated at 7 days, but the strength continued to increase. As shown in Fig. 6, the strength of the dry-cured concrete was 4990 psi at 28 days, with an expansion of only 0.35 percent. The fog-cured concrete reached only 3000 psi at 28 days but expanded 0.90 percent. Concretes cured under each condition contained cementing material which consisted of 75 percent portland cement and 25 percent of expansive component and were restrained for the first 7 days.

Concrete pipe under external biaxial restraint — A pipe made from concrete containing cement with 18 percent expansive component was cast and maintained under external biaxial restraint for 14 days by a ¼ in. steel shell. The concrete had a cementing material content of 8.0 sacks per cu yd, a W/C of 0.45, and contained ¾ in. maximum size aggregate. The nominal slump was 3 in. The 2-in. pipe had an outside diameter of 12 in., and a height of 12 in. Shortly after being cast, the inner form was removed, and the pipe was stored in fog; at 8 days it was stored at 50 percent relative humidity; at 14 days it was removed from storage and restraint was terminated by removal of the steel shell. A concrete pipe and a typical steel form to provide biaxial restrain are shown in Fig. 7. Observations of shell strains were made throughout this test using SR-4 rosette strain gages placed on the surface of the steel shell at mid-height and near each end of the pipe. Three lines of such gages were placed 120 deg apart. Gages at cor-

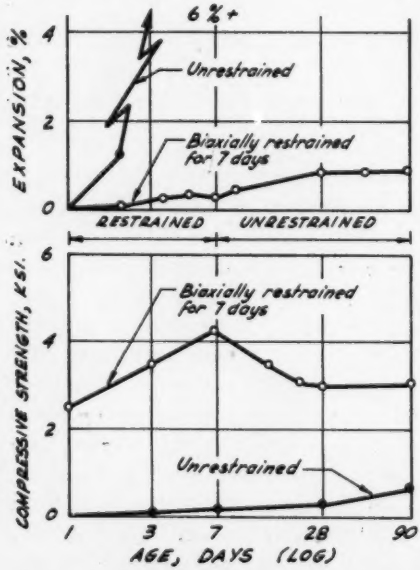

Fig. 5 — Effect of biaxial restraint on expansion and compressive strength of concrete containing expansive component. W/C = 0.45 by weight; cement content = 8.0 sacks per cu yd; 25 percent expansive component; curing at 100 percent relative humidity and 70 F

responding locations were installed also on the exposed inner surface of the concrete pipe.

As shown in Fig. 8, the circumferential stresses at the ends of the steel pipe form were high in comparison with those at midheight of the pipe form. This difference indicates that at the ends of the specimen, where the concrete had more surface area exposed to free moisture, the expansive addition was able to hydrate more rapidly and accordingly to develop greater expansion. This greater expansion near the end of the pipe caused the steel shell to bend outward near its ends, and produced a compressive stress on the outside of the steel shell.

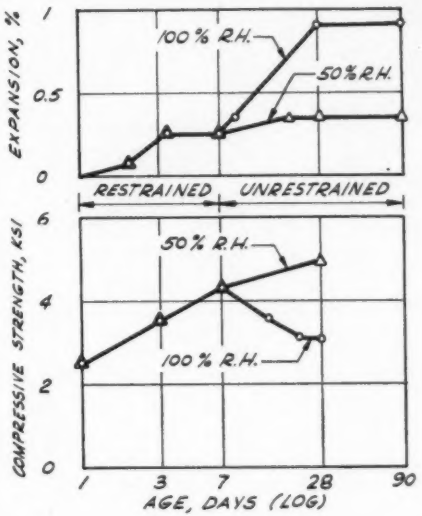

Fig. 6 — Effect of curing condition on expansion and compressive strength of concretes biaxially restrained for 7 days. W/C = 0.45 by weight; cement content = 8.0 sacks per cu yd; 25 percent expansive component

Subjecting the pipe specimen to drying at 50 percent relative humidity, subsequent to the initial 8 days of fog curing, caused about a 10 percent drop in the shell stresses of the pipe form. As shown in Fig. 8, the shell stresses after storage in dry atmosphere remained about constant, indicating that any further expansion was offset by shrinkage of the concrete.

On removal of the shell at 14 days, an immediate expansion of 40 millionths was observed from the gages on the inner surface of the concrete pipe (not shown in Fig. 8). This sudden expansion possibly was due to the release of the high restraining force caused by the shell. However, it has been noted that where expansion has not been completed, these con-

Fig. 7 — Concrete pipe and a typical steel form used for external biaxial restraint. Concrete pipe 12 in. outside diameter, 12 in. long and 2 in. thick; expansive component 18 percent; cement content = 8 sacks per cu yd; W/C = 0.45 by weight; steel form 1/4 in. thick.

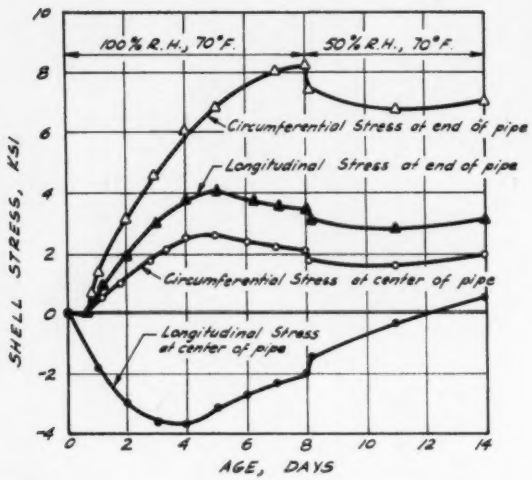

Fig. 8 — Shell stresses in steel form of concrete pipe due to biaxial restraint of concrete. $W/C = 0.45$ by weight; cement content = 8 sacks per cu yd; 18 percent expansive component

cretes can expand even under conditions of 50 percent relative humidity. The concrete pipe freed from restraint was instrumented with strain gages on the outer surface and was stored in fog. The pipe expanded an additional 0.08 percent up to 39 days (in excess of any drying shrinkage), at which time no additional significant expansion could be observed.

At 40 days, flexure specimens were cut from the companion pipe, and these were tested at 40 days and 1 year. These specimens, $2 \times 2 \times 12$ in. were simply supported on a 10-in. span with the load applied at the center. Compressive strengths were obtained from 4 in. long cut specimens of the same cross section. These specimens also were used for determinations of elastic modulus. The results of these tests are shown in Table I.

It may be noted that between these two ages the flexural strength remained constant while the compressive strength and the modulus of elasticity increased considerably.

The pipe specimen shown in Fig. 7 was subjected to internal hydrostatic pressure of 250 psi before leakage occurred through the first crack.

TEST RESULTS OF SERIES II—CONCRETES UNDER UNIAXIAL RESTRAINT

In the second phase of this program the effect of amount of expansive component and of water-cement ratio on compressive strength and on expansive characteristics of concretes subjected to external uniaxial restraint were studied. Also evaluated was the effect of a water-reducing retarder, used in amount of 4 ounces per sack of cementing material. The effect of different degrees of expansion on compressive strength was determined both for unrestrained and for uniaxially restrained concrete specimens.

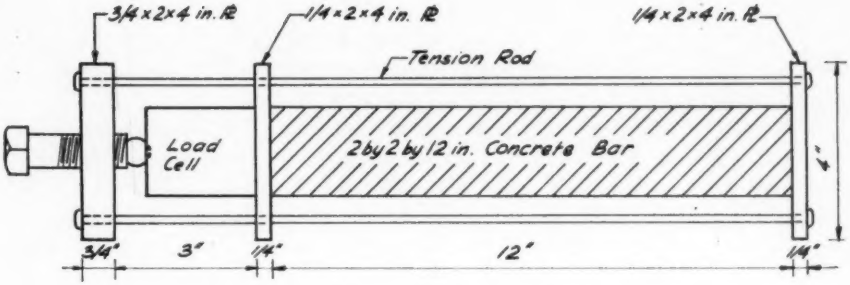

Fig. 9 — Uniaxially restrained concrete bar and frame

All concrete specimens for this phase were prismatic bars $2 \times 2 \times 12$ in. Free expansions were measured with dial-gage extensometers. For compression tests, each bar was sawed into three prisms $2 \times 2 \times 4$ in.

For uniaxially restrained concretes, bars were externally restrained by two end plates and four tension rods as illustrated in Fig. 9.

Unrestrained control concretes

Effect of amount of expansive component — The concretes used for evaluating the effect of different amounts of expansive component on free expansion and compressive strength in each case contained 7 sacks of cementing material per cu yd. The amounts of expansive component were 12.5, 17.5, 22.5, and 25 percent by weight of the total cementing material. W/C throughout was 0.425, and the concrete contained ½ in. maximum size aggregate. The nominal slump was about ¾ in. All concrete specimens were fog cured at 70 F. Results to 90 days are shown in Fig. 10. Free expansions of concretes containing cementing material with 12.5 and 25 percent expansive component was 0.16 and 1.2 percent, respectively. All concretes, regardless of amount of expansive component,

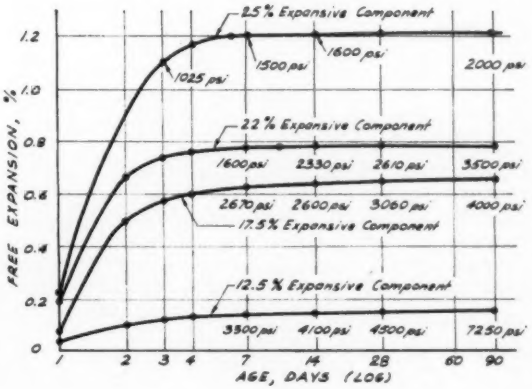

Fig. 10 — Effect of amount of expansive component on free expansion of concretes. W/C = 0.425 by weight; cement content = 7.0 sacks per cu yd; curing at 100 percent relative humidity and 70 F; compressive strength of 2 x 2 x 4-in. prisms

exhibited rapid expansion during the first 3 days and a stabilization of expansion at 5 or 6 days. Additional expansions after about 7 days were low for the low-expansion concretes and negligible for the high-expansion concretes.

Compressive strengths of the several concretes at 7, 14, 28, and 90 days are given in Fig. 10. General reductions of compressive strengths were observed with increased expansions. Concrete which expanded 1.2 percent developed a compressive strength of only 1600 psi at 14 days, as compared with concrete of 0.16 percent expansion and 4100 psi at the same age.

The effect of free expansion on compressive strength is shown in Fig. 11 for concretes at 7, 28, and 90 days. Also shown are the compressive strengths for the control concretes without expansive component. The plotted data are the average values for concretes containing from 12.5 to 25 percent of expansive component and 7.0 sacks of cementing material per cu yd. W/C ranged from 0.385 to 0.425. Although the compressive strength decreased with increase in expansion, concretes which expanded as much as 0.5 percent developed compressive strengths suitable for structural concrete, being about 2000 psi at 7 days, 3500 psi at 28 days, and 5500 psi at 90 days. The relationship between free expansion (log scale) and compressive strength exhibited by these data (Fig. 11) is similar to that demonstrated in Fig. 4 for concretes of fixed cement content and W/C. It is to be noted that an expansion in excess of 0.3 to 0.4 percent results in a relatively large decrease in compressive strength, especially at later ages (28 and 90 days).

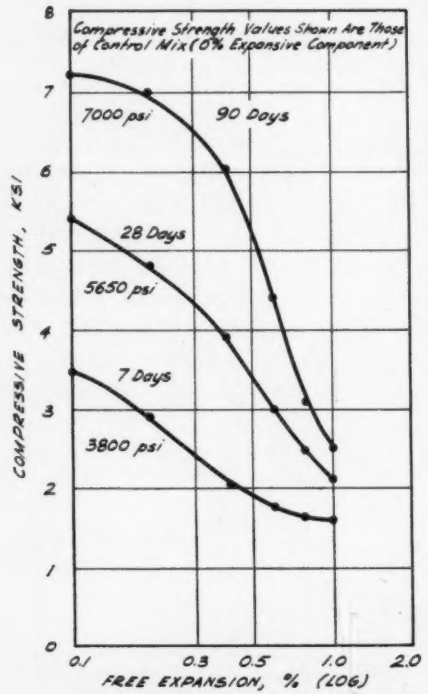

Fig. 11 — Effect of free expansion on compressive strength of concrete cured in fog at 70 F. Average values for concrete having $W/C = 0.385$ to 0.425 by weight; cement content = 7.0 sacks per cu yd; 12.5 to 25 percent expansive component; curing at 100 percent relative humidity and 70 F

The curves of Fig. 11 could be extended to include, for compressive strengths at various ages, free expansions for mixes over a wider range of cement contents, W/C ratios, and percentages of a given expansive component. Likewise, simi-

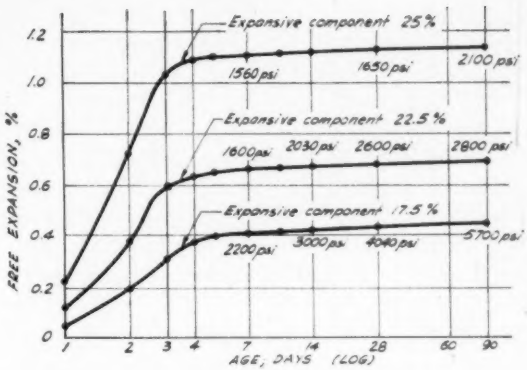

Fig. 12 — Effect of amount of expansive component on free expansion of concretes. W/C = 0.385 by weight; cement content = 7.0 sacks per cu yd; water-reducing retarder 4 ounces per sack; curing at 100 percent relative humidity and 70 F; compressive strength of 2 x 2 x 4-in. prisms

lar charts can be prepared for expansion of prismatic bars under restraint. A separate chart could also be prepared for each of several percentages of restraining steel. Inasmuch as the magnitude of the restrained expansion governs the magnitude of self-stress, in both steel and concrete, such charts could be used to select the mix to produce the required expansion under a given degree of restraint in concrete of desired compressive strength.

The design of the trial mix could then be obtained from charts containing the data used in the construction of the curves relating compressive strengths and restrained expansion. The trial mix may then readily be evaluated in the "calibration" restraining apparatus (Fig. 9).

In a corresponding series of tests the use of a water-reducing retarder in the amount of 4 ounces per sack of total cementing material permitted the reduction of W/C from 0.425 to 0.385. In Fig. 12 are shown free expansions and compressive strengths, up to 90 days, for concretes having a cementing material content of 7.0 sacks per cu yd and containing 17.5, 22.5, and 25 percent of expansive component. The expansion characteristics are similar to those observed for the higher W/C ratio concretes (Fig. 10), as are the higher compressive strengths and the magnitudes of expansion.

To evaluate the effect of high percentages of expansive component on the properties of concretes, three groups of specimens were cast from concretes containing respectively 20, 23, and 26 percent of expansive component by weight of total cementing material. The total cementing material contents of these concretes were 7.5, 7.8, and 8.1 sacks per cu yd, respectively, and the W/C ratios were 0.35. Again the maximum size of aggregate was ½ in. and the concretes had a slump of about ¾ in. Fig. 13 shows the free expansions and compressive strengths of these fog-cured concretes up to 90 days. The tendency for the free expansion to stabilize at 5 or 6 days is similar to the results obtained in the tests on the lower percentages of expansive component (Fig. 10 and 12). Concrete bars

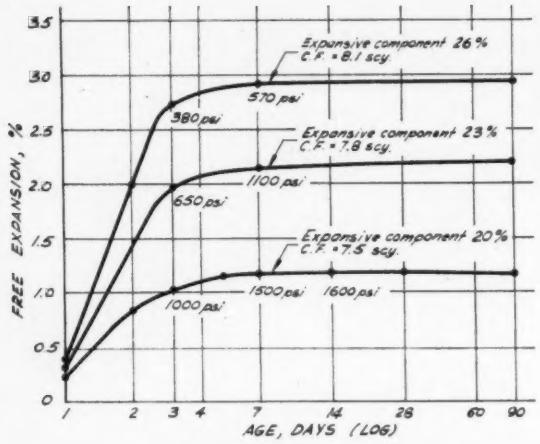

Fig. 13 — Effect of high percentages of expansive component on free expansion of concretes. $W/C = 0.35$ by weight; curing at 100 percent relative humidity and 70 F; compressive strength 2 x 2 x 4-in. prisms

made with cement containing 26 percent of expansive component exhibited almost 3 percent free expansion. Although the compressive strength of this concrete was only 570 psi at 7 days, no disruption of the bars had occurred.

Effect of water-cement ratio — Two groups of specimens were cast to determine the effect of W/C on the expansion characteristics of unrestrained concretes. One group of concretes was made with cement containing 23 percent of expansive component and the other 26 percent. The W/C ratios employed were 0.30 and 0.35 for concrete with 26 percent of expansive component and 0.35, 0.40, and 0.45 for concretes with 23 percent of expansive component. These concretes had a cementing material content of 8.1 and 7.8 sacks per cu yd, respectively; all con-

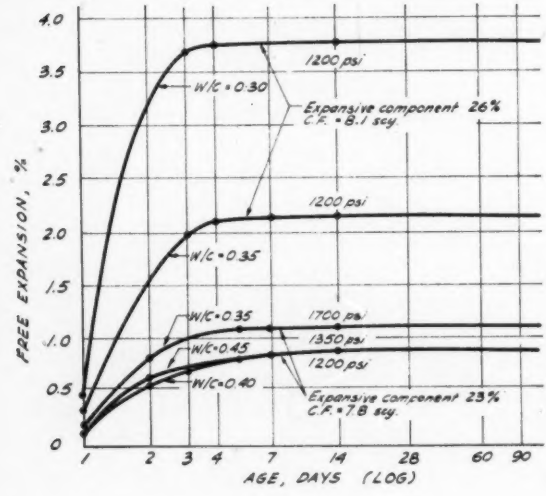

Fig. 14 — Effect of W/C ratio on free expansion of concretes containing expansive component. Compressive strength of 2 x 2 x 4-in. prisms; curing at 100 percent relative humidity and 70 F

tained ⅜ in. maximum size aggregate, and all were continuously fog cured. The results of these tests are shown in Fig. 14. Also shown are the compressive strengths at 14 days. In general, the lower the W/C ratio the higher the free expansion. Apparently the lower the W/C ratio the higher the concentrations of dissolved salts; hence the greater the assurance that the liquid phase continues in a condition of supersaturation with respect to lime. For concretes made with cement containing 26 percent of expansive component, reduction of the W/C ratio from 0.35 to 0.30 almost doubled the expansion (from 2.2 to 3.8 percent). However, the compressive strength of the concrete prisms with a W/C of 0.30 was no greater for corresponding concrete having the higher ratio of 0.35.

Free expansions for concretes with W/C ratios of 0.40 and 0.45 were about equal. However, with further reduction of W/C ratio to 0.35 free expansion as well as compressive strength were increased.

These data illustrate the importance of low W/C in producing higher expansions together with satisfactory compressive strength.

Uniaxially restrained expansive concrete specimens

The tests on uniaxially restrained concrete specimens were conducted in a manner to determine the stresses developed in the restraining steel rods and in the concrete due to self-stress, or more properly, chemical prestress. For this group of tests, concretes were used having different W/C ratios and different richnesses of mix.

Fig. 9 shows a diagram of the restraining system and of the load cell used to measure self-stress forces as they developed. The steel rods of the restraining frame were of such sizes as to furnish the desired percentage of reinforcement. This restraining mechanism provided means by which any combination of conditions involving the factors influencing expansion and resulting self-stress could be calibrated or evaluated for purposes of design. Concrete bars were stripped at 6 hr after casting and were inserted in the restraining frame; then the two end plates were brought into contact with the bar with the adjusting screw. The assembly was then stored in fog at 70 F. As the concrete expanded, tension

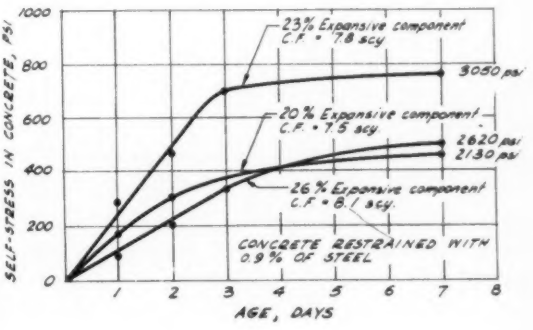

Fig. 15 — Effect of amount of expansive component on development of self-stress — uniaxial restraint. W/C = 0.35 by weight; curing at 100 percent relative humidity and 70 F; compressive strength of 2 x 2 x 4-in. prisms

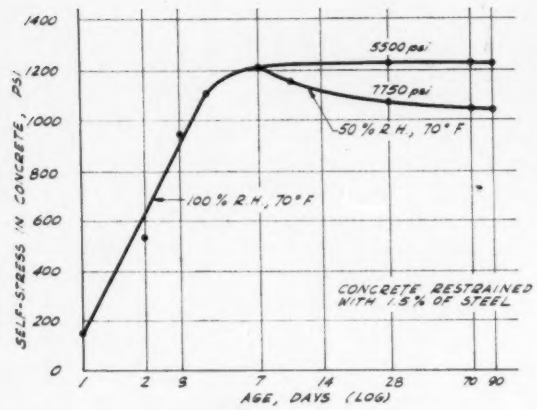

Fig. 16 — Loss of self-stress due to drying shrinkage of uniaxially restrained concrete. W/C = 0.30 by weight; cement content = 8.5 sacks per cu yd; 30 percent expansive component; compressive strength of 2 x 2 x 4-in. prisms

was developed in the steel rods, and self-stress compression was developed in the concrete.

Effect of amount of expansive component — In preliminary tests a concrete mix was employed that would develop a free expansion of 0.8 percent in the hardened concrete. Cured in fog for 10 days, and restrained with 0.9 percent of steel, this concrete developed a stress of about 20,000 psi in the steel. However, when subsequently stored at 50 percent relative humidity, these stresses were almost completely relieved. It was indicated, therefore, that concretes capable of developing high free expansions would be required for adequate levels of self-stress.

The concretes selected for study of high expansive concretes contained 20, 23, and 26 percent of expansive component with a W/C ratio of 0.35 by weight. Fig. 15 shows the compressive stresses developed in the restrained concrete bars and the compressive strengths at 7 days. Concrete containing 23 percent of expansive component developed the highest self-stress. The self-stress of this concrete at 7 days was 760 psi, which corresponds to 84,000 psi tensile stress in the steel. The 7 day compressive strength of this concrete was 3000 psi, which is high for such an early age. The self-stress developed in the concretes containing 20 and 26 percent expansive component was about 500 psi.

It should be pointed out that the 23 percent of expansive component is not necessarily the optimum percentage for maximum self-stress of concretes having a W/C ratio of 0.35 by weight. For each given percentage of expansive component, there will be an optimum W/C ratio. Data indicating these optimum values are as yet not available.

Loss of self-stress due to drying shrinkage — Loss of self-stress due to drying shrinkage was measured for a concrete containing 30 percent of expansive component and with a W/C of 0.30. This low W/C ratio was obtained using a water-reducing retarder in the amount of 4 ounces per sack of cement. As shown in Fig. 16 the concrete, restrained with 1.5

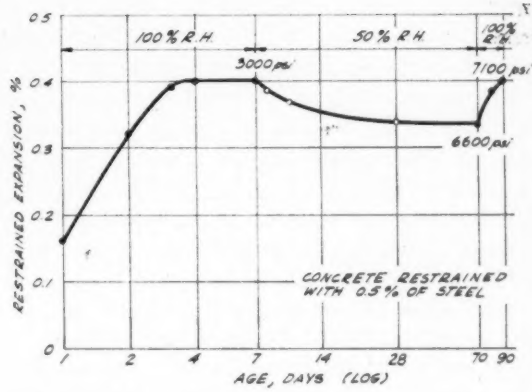

Fig. 17 — Effect of humidity on volume change of uniaxially restrained concrete at 70 F. W/C = 0.30 by weight; cement content = 8.5 sacks per cu yd; 30 percent expansive component; compressive strength of 2 x 2 x 4-in. prisms

percent of steel, developed 1200 psi self-stress at 7 days, corresponding to a tensile stress of 80,000 psi in the steel. At 7 days a group of these specimens was moved to storage at 50 percent relative humidity to evaluate loss of self-stress due to drying shrinkage. After 83 days of storage, the bars showed loss in self-stress amounting to only 190 psi, or about 16 percent. The compressive strength of this concrete at 28 days was 7750 psi, while for corresponding fog cured specimens it was only 5500 psi at the same age. This large difference in compressive strengths is due primarily to the inverse relationship between expansion and compressive strength.

Data of Fig. 16 indicate that the expansion was almost complete at 7 days (90 percent at 4 days) and that at 50 percent relative humidity about 3 months was required for the bars to come to equilibrium. The authors have found that where expansion occurs rapidly in fog, volumetric equilibrium is reached more quickly during drying.

Effect of richness of mix — In another test the cementing material content of the concrete containing 30 percent of expansive component was increased to 10.0 sacks per cu yd, with the W/C ratio correspondingly reduced to 0.29. The concrete prisms were restrained with the equivalent of 2.0 percent of steel. After 14 days of fog curing, self-stress in the concrete was 1600 psi, which corresponds to a tensile stress in the steel of over 80,000 psi. The compressive strength of the self-stressed concrete at 14 days was 4200 psi. These results compared with those just discussed indicate that increase in cement content and reduction in W/C ratio produces higher values of self-stress.

Effect of storage humidity on volume change — One set of restrained concrete bars was subjected to a curing cycle consisting of storage at 70 F at the following relative humidities: 100 percent for 7 days, 50 percent from 7 to 70 days, and back to 100 percent at 70 days. Observed volume changes are shown in Fig. 17. The concrete had a total cementing

material content of 8.5 sacks per cu yd (containing 30 percent expansive component) and a W/C of 0.30. The concrete was restrained with 0.50 percent of steel.

It may be noted that during the curing at 100 percent relative humidity the concrete expanded rapidly and became stabilized at about 0.4 percent expansion at 4 days. During storage at 50 percent relative humidity from 7 to 70 days, 0.07 percent shrinkage was observed. However, when the concrete bars were returned to storage at 100 percent relative humidity, a complete recovery of the shrinkage was obtained within 30 days. At 7 days the self-stress in the concrete was 630 psi, corresponding to a stress in the steel of 126,000 psi, and its compressive strength was 3000 psi. At 70 days (after 63 days at 50 percent relative humidity) the self-stress in the concrete was 535 psi, corresponding to a steel stress of 107,000 psi. During storage at 100 percent relative humidity (from 70 to 90 days), the recovery of shrinkage also restored the self-stress to approximately the same values as were observed at 7 days. The 90 day compressive strength was 7100 psi.

It appears that practically all of the creep had been taken out of this concrete, as indicated by the fact that it returned to original dimensions and stress conditions when stored in fog after being permitted to shrink due to drying.

During expansion this material appears to have properties similar to a plastic body, since it continues to deform with application of expansive forces and retains its form after completion of the expansion. Such conditions are significantly appropriate for creep in compression. If this is true, then the application of prestress to post-tensioned conventional concrete elements immediately after final set, with the stress-strength ratio held constant with increase in strength with age, the creep characteristics of the concrete would be substantially removed in a short period.

DISCUSSION OF RESULTS

The expansive component here investigated appears to have the necessary characteristics needed to produce a controllable self-stressed concrete. In practice, to insure complete control of composition and fineness, it would be necessary to prepare an expansive cement at a cement plant where the portland cement component could be "tailored" to best performance of the expansive cement. The expansive component would be combined with the portland cement either as a blend of separately ground components or, preferably, interground. A uniform distribution of the expansive component throughout the matrix of portland cement is essential. In all probability, such conformity could not be obtained with current rapid practices of mixing concrete, if the expansive component is used as an admixture.

The W/C ratio has a significant effect on the expanding characteristics of a concrete mix. Within the range of tests herein reported, the lower the W/C ratio the greater was the expansion, other things being equal.

A retarding water-reducer appeared to affect the expansive characteristics of a concrete only through the resulting reduction in W/C. The reduction in W/C improved compressive strength and to some degree the expansive characteristics.

Curing conditions play an important role in the properties of expansive cements. The availability of water governs the rate of expansion. Lack of water can reduce the magnitude of ultimate expansion. In practice the use of delayed accessibility of external curing water might be desirable to achieve adequate strength before development of high expansion under restrained conditions. A practically ultimate expansion can be accomplished under relatively short curing periods, provided adequate strength is first developed. The concretes investigated developed practically their full expansion after 4 days of fog curing.

All tests here reported were performed at a temperature of 70 F. The effects of lower or of higher temperatures upon the expansive reaction have not been investigated.

CONCLUSIONS

The conclusions drawn are pertinent only to the conditions of tests and the material employed in this investigation (Type I portland cement; expansive component calcium sulfoaluminate; 12.5 to 30 percent expansive component; aggregates of normal composition; W/C ratios between 0.29 to 0.45 by weight; cementing material content 7.0 to 10.0 sacks per cu yd; curing at 70 F and 50 or 100 percent relative humidity; external biaxial or uniaxial restraint). From the results obtained the following generalizations can be made.

1. Either unrestrained or restrained expansions can be developed in concretes within ranges practical for structural concretes with the calcium sulfoaluminate expansive component employed in this investigation. Restraint of expansion develops compressive stress in the concrete.

2. For a given curing condition, at any age the greater the unrestrained expansion the lower the compressive strength.

3. Even with the reductions in strength which accompany free expansion, it is possible with adequate restraint to produce self-stressing concrete of strength suitable for structural work. The greater the restraint the greater the strength.

4. For the concretes investigated, the higher the proportion of expansive component in the cementing material the larger the ultimate expansion.

5. Magnitude of self-stress is affected not only by the amount of expansive component but also by the W/C ratio of the mix, the curing conditions, and the percentage of steel employed in restraining the expansion.

ACKNOWLEDGMENTS

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Propiedades de un cemento de expansión para la prefatiga química

Las características de expansión y las resistencias a la compressión fueron determinadas para los concretos que contienen cemento de expansión, para evaluar los factores relativos al control de la reacción de expansión. El cemento de expansión consiste en un componente de cemento Portland y un componente de la adhidrita de sulfoaluminato de calcio. Los factores que tienen influencia en la magnitud y velocidad de la reacción de expansión comprenden: la composición química de los componentes, la fineza del componente del sulfoaluminato, las proporciones de los dos componentes en el aglutinate total, la relación del agua al aglutinate total, la riqueza de la mezcla, las condiciones de curación, y el grado de restricción.

En estas investigaciones, la composición química de los componentes y la fineza del componente del sulfoaluminato se mantuvieron esencialmente constantes, y los otros factores, que influyen en la magnitud y la velocidad de la expansión fueron variodas.

Para este estudio, la restricción en todos los casos se proporcionó por medio de mecanismos exteriores de acerom en unos casos siendo la restricción uniaxial, y en otros casos biaxial. En ausencia de la restricción, los concretos manifestaron expansiones libres hasta 6 por ciento o más.

Pas propiedades de los concretos dadas en este documento indican que ciertos cementos de expansión que fueron ensayados son convenientes para un trabajo estructural y pueden emplearse eficazmente en la fabricación de los miembros prefatigados químicamente bajo condiciones de restricción exterior. Se estableció que con las proporciones de mezla y curación adecuadas, con las composiciones, proporciones, y fineza de los componentes fijas, es posible producir concretos que tengan las características predeterminadas deseadas dentro de un límite práctico.

Les caractéres d'un ciment expansif pour la mise sous précontrainte par voie de la chimie

Les caractéres de détente et les résistances á la compression étaient déterminés pour les bétons comprenant un ciment expansif, pour pouvoir évaluer les facteurs intervenant dans le contrôle de la réaction expansive. La ciment expansif comprend un élément de ciment de Portland ainsi qu'un élément d'anhydrite de calicum sulphoaluminate. Les facteurs gouvernant l'amplitude et la vitesse de la réaction expansive comprennent: la composition chimique des éléments, la finesse de l'élément sulphoaluminate, les proportions des deux éléments dans l'ensemble de la matiére du ciment, le rapport de l'eau á l'emsemble de la matiére du ciment, la richesse du mélange, les conditions de la maturation et le degré de restreinte.

Au cours de cette investigation, la composition chimique des éléments et la finesse de l'élément sulphoaluminate sont restées essentiellement constantes, et d'autres facteurs agissant sur l'amplitude et la vitesse de la détente ont été modifiés.

Pour cette étude, la contrainte était toujours fournie par des mécanismes extérieurs en acier, la contrainte étant soit uniaxiale soit biaxiale. En l'absence de contrainte, des bétons ont présenté une expansion libre de jusqu'á 6 pour cent et plus.

Les caractères des bétons rapportés ici indiquent que certains des ciments ex-

pansifs essayés sont aptes pour les ouvrages de construction et qu'ils peuvent être utilisés avec succès dans la fabrication d'organes précontraints chimiquement sous des conditions de contrainte externe. Il a été établi qu'avec un mélange judicieusement proportionné et maturé, et avec les compositions, les proportions et la finesse des éléments déterminées, il est possible de produire des bétons ayant des caractères prédéterminés voulus dans des limites pratiques.

Eigenschaften eines dehnbaren Betons fuer chemische Vorspannung

Die Ausdehnungseigenschaften und Druckfestigkeitswerte wurden fuer Betone mit dehnbarem Zement bestimmt, um die Faktoren, die sich auf die Regelung der Ausdehnungsreaktion beziehen, abzuschaetzen. Dehnbarer Zement besteht aus einer Portland Zement Komponente und einer Kalzium-Sulfoaluminat Anhydrit-Komponente. Die Faktoren, die das Ausmass und die Geschwindigkeit der Ausdehnungsreaktion beeinflussen, umfassen: chemische Zusammensetzung der Komponenten, Feinheit der Sulfoaluminat Komponente, Anteil der beiden Komponenten am gesamten Zement-Material, Verhaeltnis von Wasser zum gesamten Zement-Material, Reichtum der Mischung, Nachbehandlungsbedingungen und Schrumpfungsgrad.

In dieser Untersuchung wurden die chemische Zusammensetzung und die Feinheit der Sulfoaluminat-Komponente im Wesentlichen konstant gehalten, und andere Faktoren, die das Ausmass und die Geschwindigkeit der Ausdehnung beeinflussen, wurden geaendert.

Fuer diese Untersuchung wurde Schrumpfung in allen Faellen durch Stahleinrichtungen vorgesehen. In manchen Faellen war die Schrumpfung einachsig, und in anderen Faellen zweiachsig. In der Abwesenheit von Schrumpfung, zeigten die Betone freie Ausdehnungen bis zu 6 prozent und mehr.

Die Eigenschaften des hier beschriebenen Betons zeigen, dass gewisse dehnhafte Zemente fuer Bauzwecke geeignet sind, und dass sie wirkungsvoll in der Herstellung von chemisch vorgespannten Gliedern unter Bedingungen von aeusserer Schrumpfung verwendet werden koenne.

Es wurde gefunden, dass er mit der richtigen Bemessung der Bestandteile der Mischung und mit der richtigen Nachbehandlung, wobei dei Zusammensetzung, die Anteile und die Feinheit der Komponenten festgesetzt sind, moeglich ist, Betone herzustellen, die die gewuenschten, vorbestimmten Eigenschaften innerhalb einer praktischen Reichweite haben.

Title No. 58-4

Design Constants for Interior Cylindrical Concrete Shells

By A. L. PARME and H. W. CONNER

To increase confidence in the use of the beam method for the analysis of interior cylindrical shells and also provide data for those cases in which the beam method is not applicable, design constants for interior cylindrical shells based on the shell theory are presented. These constants enable the ready evaluation of the internal forces in shells by a few simple multiplications. In addition studies on the effect of the geometry of the shell on the longitudinal distribution of stresses are included. A discussion of the effect of continuity enlarges the applicability of the design constants.

■ DESIGN CONSTANTS based on the assumption that the shells behave like a beam were presented by the authors in their discussion of the article "Cylindrical Shell Analysis Simplified by Beam Method" by James Chinn.[1] Those constants provided a convenient method of readily evaluating the internal forces and moments created by uniform dead loads in long and in intermediate length cylindrical shells. While those constants are perfectly satisfactory for long shells and were recommended in this range, some vagueness regarding the applicable limit for intermediate length shells existed.

The uncertainty of applicable limit was caused primarily by the validity of the assumption that linear strain depends not only on the ratio of radius to longitudinal span but also on the subtended angle and the ratio of thickness to radius. Because of the interdependence of the effect of these factors, no precise limits for the beam method could be given.

To remove the uncertainty and at the same time reduce the labor involved in the design of cylindrical shells which cannot be adequately treated by the beam method, a new series of comparable constants is presented in Table 1. These constants have been computed on the basis of the shell theory given in ASCE *Manual* No. 31.[2] Consequently these newer constants are a function of r/L and r/t as well as the subtended angle, ϕ_k.

To avoid interpolation as much as possible, values are given for r/t values of 100, 200, and 300 and for six values of r/L varying from 0.4

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to 2.6. For ϕ_k less than 45 deg, it was found that the modified beam method was sufficiently accurate for all values of r/L less than 0.6. Thus for the portion of Table 1 dealing with ϕ_k less than 45 deg, the internal forces are only given for values of r/L greater than 0.6. When ϕ_k is greater than 45 deg, it was found necessary to include an r/L as low as 0.4 to provide a good transition from values as computed by the beam method to those computed by shell theory.

It should be noted that although values are tabulated for $r/t = 300$, which represents a shell beyond practical limit, they have been included to avoid extrapolation for cases of r/t beyond 200. Likewise, the selection of $r/L = 2.6$ represents an arbitrary limit. The internal forces are concentrated near the edge for values of r/L greater than those listed. For this reason, the arrangement of Table 1 is not suitable for such shells.

Values have been given only for load varying as the dead weight because numerous comparisons made with different r/L values indicate that the effect of a uniform load could be closely approximated by an equivalent dead weight expressed:

$$p_s = p_u \left(\frac{\sin \phi_k}{\phi_k} \right) \quad (1)$$

The constants have been determined on the basis that transverse rotation and horizontal displacement of the longitudinal edges of the shell are prevented. They are thus applicable to interior barrels in which restraint to such movement is provided by adjacent barrels. However, they can be applied with tolerable accuracy to the interior half of the exterior bay since the effect of disturbance of loads on the far edge has only minor influence on the first interior valley. This is especially true, since to prevent excessive deflection of the free edge an edge beam should always be provided (except for long shells with short chord width) at the exterior edge.

Determination of the internal forces in cylindrical shells subjected to uniform, longitudinal loading by the shell theory requires that the actual load be approximated as the sum of partial loads varying sinusoidally according to a Fourier series in the longitudinal direction. Gen-

erally only the first or at most two partial loads are used; with adjustments made especially to the shear value on the basis of statical requirements. However, since Table 1 was prepared using an electronic computer, the algebraic sum of four partial loads was used to avoid adjustment. Even with this number of loads, to achieve sufficient accuracy, it was found necessary in some cases to employ Euler's convergence technique.[3] However, such care should not be interpreted as needed or justified on the basis of underlying assumptions. Its worth rests solely on the fact that it permitted a more accurate comparison of values as the parameters r/t and r/L are varied, and enabled a more precise examination of the variation of the internal forces in the longitudinal direction.

In this connection, the constants in Table 1 give only the transverse distribution of forces at midspan and at the support with no indication of the longitudinal distribution of forces. The reason for this is that the exact expression for longitudinal distribution, even for simply supported shells, is highly complex involving four functions. Fortunately within the range of the tabulated values the longitudinal distribution can be approximated by a well recognized relationship.

For example, the distribution of T_x closely follows a parabolic distribution of a uniformly loaded beam, as might be expected (see Fig. 1). This is true for widely different shells represented by r/L equal to 0.6 and 2.6. Although the curves shown in Fig. 1 have been computed on the basis of $\phi = 27.5$ deg, they are typical of those for other angles. A sinusoidal distribution of T_x would also be satisfactory.

For design purposes T_ϕ can be assumed to be uniform in the longitudinal direction as can be inferred from Fig. 2. Because the analysis

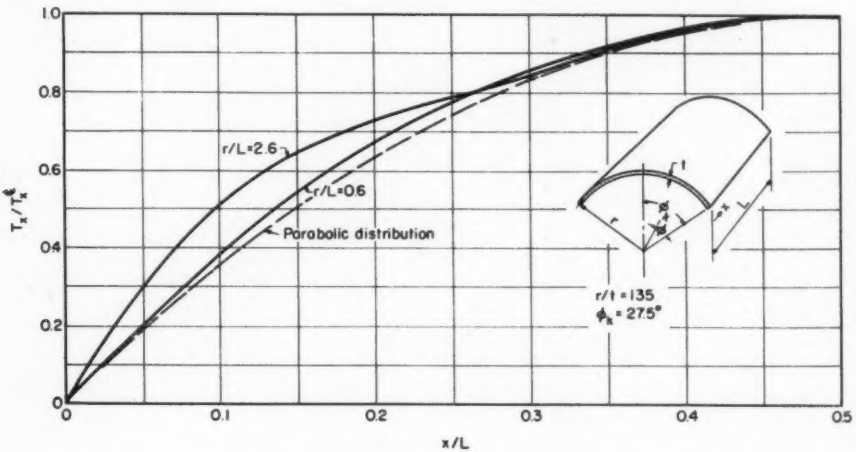

Fig. 1—Longitudinal distribution of T_x at valley

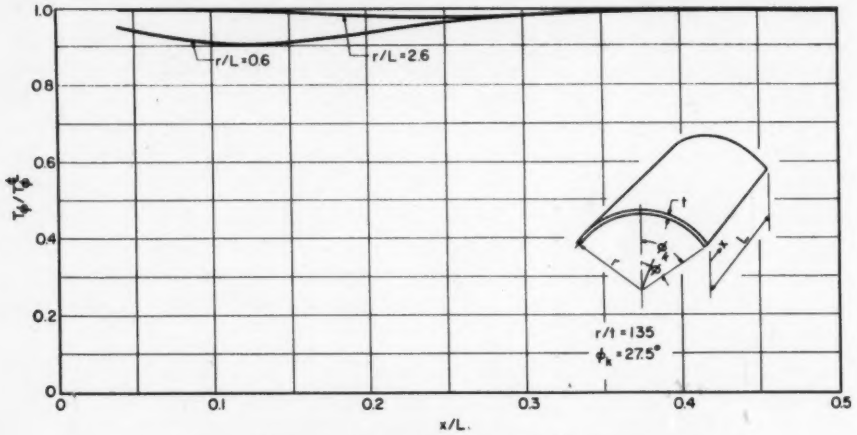

Fig. 2—Longitudinal distribution of T_ϕ at crown

has been based on the prescribed boundary condition that the shell is supported by a rigid member at $x = 0$ and $x = L$, the value of T_ϕ decreases theoretically to zero at the support. The transition from zero to the full value, however, takes place over a very short interval. Thus, especially for values near the crown, the assumption of a uniform distribution of T_ϕ is justified. Distribution of T_ϕ in the valley can also be considered uniform even though a careful evaluation of the distribution in this area indicates some departure from uniform distribution near the support. The computed variation near the support may be due to the sensitivity of the results to the number of load terms used. This is so because the absolute value is generally quite small compared

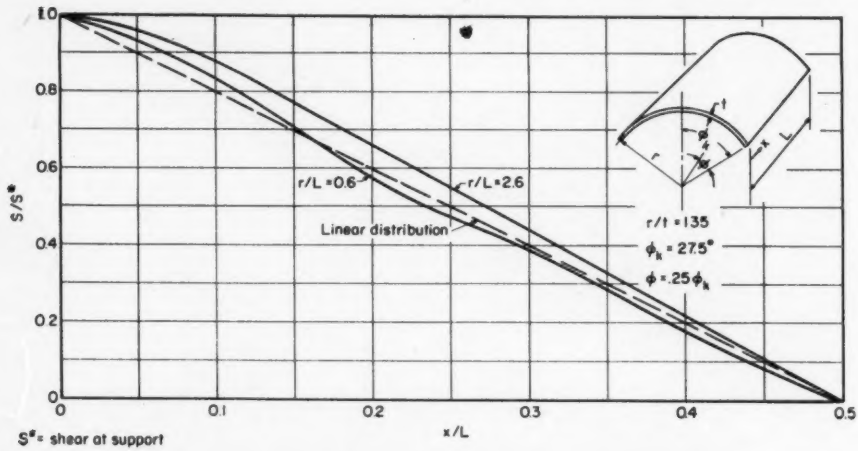

Fig. 3—Longitudinal distribution of shear at $\phi = 0.25\,\phi_k$

to the crown value with the final result equal to the difference of almost like values. Because the values are small and have almost no effect on the design, the assumption of uniform distribution of T_ϕ in the valley is justified.

As in the case of the distribution of the T_x forces, the distribution of shear can be assumed to be like that in a beam with the shear varying linearly from a maximum value at the support to zero value at midspan. As shown in Fig. 3, the distribution as computed by the shell theory gives slightly higher values, but the variation from the linear distribution is negligible.

There is one important aspect of shear distribution which warrants some comment. As shown in Fig. 4, in which a plot of the transverse distribution of shear at various sections along the shell are superimposed on each other, the shear tends to be concentrated towards the valley as the support is approached. From this plot it should not be inferred that the magnitude of shear does not decrease proportionally to the distance from the support. For clarity in presentation of the variation in transverse distribution, all values have been plotted in terms of the value of shear at $\phi = 0.5\,\phi_k$; hence, all values are relative. While this change in transverse shear distribution is insignificant with respect to its effect on the direct stresses in the shell, it has a pronounced effect on the longitudinal moment distribution.

As in the case of T_ϕ, the boundary condition of supports rigid in the transverse direction leads to zero moment at the support. For long shells, as discussed in Reference 1, the moment increases at a variable rate from zero at the support to a maximum value near the quarter

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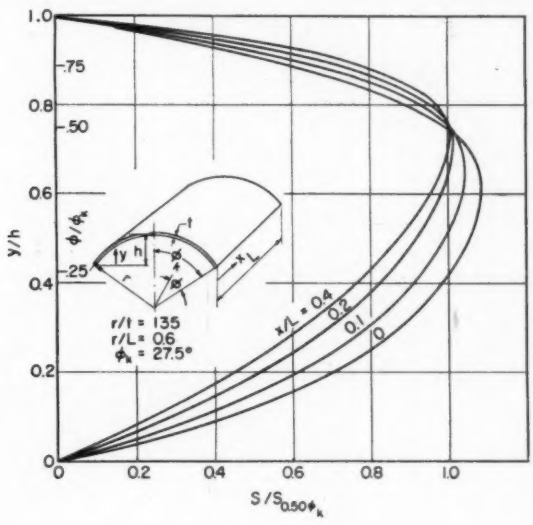
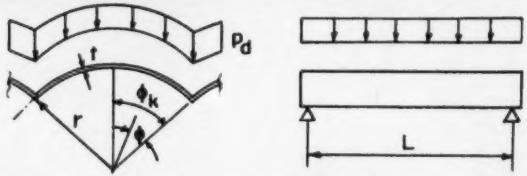


Fig. 4—Relative distribution of shear

TABLE I — INTERNAL FORCES IN A MULTIPLE CYLINDRICAL SHELL DUE TO DEAD LOAD

$T_x = -\frac{L^2}{r}$ [p_d col. (1)] $\qquad S^* = -L$ [p_d col. (3)]

$T_\phi = r$ [p_d col. (2)] $\qquad M_\phi = r^2$ [p_d col. (4)]



r/L	ϕ	r/t = 100 T_x (1)	r/t = 100 T_ϕ (2)	r/t = 100 S (3)	r/t = 100 M_ϕ (4)	r/t = 200 T_x (1)	r/t = 200 T_ϕ (2)	r/t = 200 S (3)	r/t = 200 M_ϕ (4)	r/t = 300 T_x (1)	r/t = 300 T_ϕ (2)	r/t = 300 S (3)	r/t = 300 M_ϕ (4)
													$\phi_k = 22.5$ deg
	$1.00\phi_k$	−5.976	−1.415	0.000	−0.00289	−6.035	−1.452	0.000	−0.00315	−6.011	−1.476	0.000	−0.00328
	$0.75\phi_k$	−4.893	−1.187	1.959	−0.00102	−4.946	−1.210	1.882	−0.00118	−4.947	−1.219	1.787	−0.00123
0.6	$0.50\phi_k$	−1.591	−0.597	3.258	0.00251	−1.615	−0.598	3.227	0.00256	−1.648	−0.583	3.160	0.00263
	$0.25\phi_k$	4.086	0.077	3.034	0.00234	4.129	0.083	3.126	0.00251	4.123	0.099	3.189	0.00258
	0	12.395	0.390	0.000	−0.00742	12.539	0.393	0.000	−0.00729	12.608	0.398	0.000	0.00738
	$1.00\phi_k$	−5.682	−1.430	0.000	−0.00286	−5.461	−1.471	0.000	−0.00307	−4.934	−1.471	0.000	−0.00302
	$0.75\phi_k$	−4.777	−1.193	1.783	−0.00102	−4.760	−1.219	1.624	−0.00117	−4.626	−1.225	1.436	−0.00118
1.0	$0.50\phi_k$	−1.808	−0.589	3.094	0.00248	−2.069	−0.586	2.998	0.00248	−2.510	−0.593	2.854	0.00242
	$0.25\phi_k$	3.853	0.087	3.117	0.00235	3.771	0.100	3.247	0.00251	3.531	0.097	3.360	0.00251
	0	13.013	0.393	0.000	−0.00743	13.564	0.396	0.000	−0.00727	14.339	0.394	0.000	−0.00715
	$1.00\phi_k$	−5.103	−1.420	0.000	−0.00252	−4.174	−1.419	0.000	−0.00251	−2.768	−1.360	0.000	−0.00220
	$0.75\phi_k$	−4.555	−1.198	1.592	−0.00093	−4.337	−1.214	1.299	−0.00103	−3.935	−1.206	0.939	−0.00096
1.4	$0.50\phi_k$	−2.241	−0.612	2.911	0.00221	−3.094	−0.644	2.725	0.00203	−4.260	−0.704	2.462	0.00175
	$0.25\phi_k$	3.416	0.069	3.222	0.00223	2.972	0.051	3.396	0.00229	2.255	0.002	3.594	0.00216
	0	14.172	0.386	0.000	−0.00709	15.834	0.376	0.000	−0.00661	17.999	0.358	0.000	−0.00611
	$1.00\phi_k$	−4.305	−1.382	0.000	−0.00204	−2.523	−1.326	0.000	−0.00180	−0.473	−1.224	0.000	−0.00135
	$0.75\phi_k$	−4.238	−1.189	1.374	−0.00079	−3.743	−1.196	0.911	−0.00083	−3.080	−1.175	0.408	−0.00072
1.8	$0.50\phi_k$	−2.831	−0.652	2.707	0.00181	−4.409	−0.739	2.408	0.00145	−6.124	−0.838	2.041	0.00105
	$0.25\phi_k$	2.788	0.026	3.341	0.00199	1.817	−0.040	3.572	0.00195	0.607	−0.126	3.838	0.00175
	0	15.841	0.369	0.000	−0.00648	19.067	0.344	0.000	−0.00568	22.579	0.314	0.000	−0.00493
	$1.00\phi_k$	−3.440	−1.327	0.000	−0.00154	−1.018	−1.237	0.000	−0.00119	1.149	−1.118	0.000	−0.00074
	$0.75\phi_k$	−3.868	−1.169	1.148	−0.00064	−3.113	−1.173	0.548	−0.00063	−2.283	−1.144	0.009	−0.00052
2.2	$0.50\phi_k$	−3.460	−0.691	2.492	0.00141	−5.596	−0.832	2.095	0.00095	−7.441	−0.946	1.677	0.00055
	$0.25\phi_k$	2.044	−0.027	3.456	0.00171	0.523	−0.137	3.729	0.00162	−1.056	−0.242	4.012	0.00140
	0	17.833	0.345	0.000	−0.00575	22.631	0.310	0.000	−0.00477	27.029	0.276	0.000	−0.00397

	$1.00\phi_k$	−2.647	−1.268	0.000	−0.00112	0.064	−1.164	0.000	−0.00075	1.982	−1.050	0.000	−0.00037
	$0.75\phi_k$	−3.490	−1.143	0.939	−0.00049	−2.541	−1.150	0.272	−0.00048	−1.614	−1.118	−1.226	−0.00038
2.6	$0.50\phi_k$	−4.022	−0.723	2.282	0.00107	−6.425	−0.903	1.821	0.00059	−8.098	−1.017	1.386	0.00024
	$0.25\phi_k$	1.269	−0.076	3.550	0.00145	−0.741	−0.221	3.838	0.00134	−2.611	−0.333	4.102	0.00113
	0	19.939	0.318	0.000	−0.00502	26.078	0.281	0.000	−0.00403	31.058	0.248	0.000	−0.00329

$\phi_k = 25$ deg

	$1.00\phi_k$	−4.856	−1.428	0.000	−0.00368	−4.851	−1.469	0.000	−0.00397	−4.740	−1.493	0.000	−0.00409
	$0.75\phi_k$	−3.986	−1.194	1.733	−0.00133	−4.005	−1.214	1.618	−0.00148	−3.981	−1.222	1.510	−0.00153
0.6	$0.50\phi_k$	−1.317	−0.595	2.920	0.00309	−1.359	−0.582	2.851	0.00320	−1.455	−0.564	2.765	0.00327
	$0.25\phi_k$	3.318	0.078	2.776	0.00295	3.326	0.097	2.869	0.00313	3.280	0.115	2.936	0.00320
	0	10.171	0.381	0.000	−0.00898	10.291	0.387	0.000	−0.00900	10.457	0.393	0.000	−0.00912
	$1.00\phi_k$	−4.482	−1.441	0.000	−0.00352	−3.951	−1.457	0.000	−0.00355	−3.067	−1.417	0.000	−0.00326
	$0.75\phi_k$	−3.849	−1.202	1.538	−0.00131	−3.728	−1.218	1.304	−0.00139	−3.487	−1.214	1.045	−0.00134
1.0	$0.50\phi_k$	−1.603	−0.591	2.740	0.00296	−2.078	−0.601	2.579	0.00286	−2.805	−0.641	2.377	0.00260
	$0.25\phi_k$	3.046	0.087	2.868	0.00293	2.808	0.087	3.018	0.00300	2.385	0.055	3.165	0.00289
	0	10.917	0.383	0.000	−0.00689	11.792	0.380	0.000	−0.00860	13.080	0.368	0.000	−0.00817
	$1.00\phi_k$	−3.747	−1.410	0.000	−0.00291	−2.273	−1.349	0.000	−0.00252	−0.573	−1.240	0.000	−0.00190
	$0.75\phi_k$	−3.581	−1.202	1.310	−0.00114	−3.171	−1.200	0.871	−0.00112	−2.640	−1.179	0.419	−0.00097
1.4	$0.50\phi_k$	−2.170	−0.635	2.532	0.00246	−3.428	0.710	2.233	0.00202	−4.841	−0.813	1.895	0.00149
	$0.25\phi_k$	2.520	0.048	2.981	0.00267	1.759	−0.011	3.219	0.00255	0.810	−0.099	3.463	0.00227
	0	12.361	0.368	0.000	−0.00819	14.777	0.344	0.000	−0.00730	17.536	0.314	0.000	−0.00635
	$1.00\phi_k$	−2.822	−1.349	0.000	−0.00216	−0.582	−1.225	0.000	−0.00153	1.240	−1.096	0.000	−0.00089
	$0.75\phi_k$	−3.218	−1.188	1.049	−0.00092	−2.510	−1.170	0.429	−0.00082	−1.803	−1.139	−0.087	−0.00065
1.8	$0.50\phi_k$	−2.874	−0.698	2.299	0.00188	−4.789	−0.836	1.872	0.00120	−6.341	−0.957	1.471	0.00065
	$0.25\phi_k$	1.790	−0.019	3.105	0.00230	0.446	−0.135	3.418	0.00205	−0.871	−0.245	3.685	0.00173
	0	14.357	0.342	0.000	−0.00719	18.414	0.302	0.000	−0.00588	22.035	0.267	0.000	−0.00483
	$1.00\phi_k$	−1.944	−1.279	0.000	−0.00150	0.571	−1.129	0.000	−0.00085	2.017	−1.014	0.000	−0.00035
	$0.75\phi_k$	−2.828	−1.164	0.801	−0.00070	−1.906	−1.141	0.107	−0.00059	−1.115	−1.107	−0.319	−0.00044
2.2	$0.50\phi_k$	−3.527	−0.755	2.066	0.00132	−5.703	−0.933	1.562	0.00064	−6.986	−1.044	1.151	0.00020
	$0.25\phi_k$	0.977	−0.089	3.215	0.00191	−0.857	−0.244	3.552	0.00163	−2.441	−0.355	3.788	0.00135
	0	16.587	0.313	0.000	−0.00616	21.928	0.267	0.000	−0.00476	25.992	0.234	0.000	−0.00383
	$1.00\phi_k$	−1.259	−1.215	0.000	−0.00100	1.147	−1.069	0.000	−0.00044	2.068	−0.977	0.000	−0.00010
	$0.75\phi_k$	−2.464	−1.137	0.599	−0.00052	−1.415	−1.117	−0.079	−0.00042	−0.564	−1.082	−0.403	−0.00030
2.6	$0.50\phi_k$	−4.014	−0.796	1.853	0.00091	−6.126	−0.993	1.315	0.00030	−7.013	−1.087	0.904	−0.00002
	$0.25\phi_k$	0.179	−0.147	3.291	0.00157	−2.057	−0.327	3.614	0.00131	−3.920	−0.437	3.806	0.00108
	0	18.799	0.285	0.000	−0.00525	25.115	0.241	0.000	−0.00395	29.611	0.212	0.000	−0.00319

*Shear forces are at supports, others are at midspan.

TABLE I (CONT.) — INTERNAL FORCES IN A MULTIPLE CYLINDRICAL SHELL DUE TO DEAD LOAD

r/L	φ	r/t = 100				r/t = 200				r/t = 300			
		T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)	T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)	T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)
													$\phi_k = 27.5$ deg
	$1.00\phi_k$	−4.008	−1.441	0.000	−0.00454	−3.916	−1.482	0.000	−0.00482	−3.672	−1.492	0.000	−0.00484
	$0.75\phi_k$	−3.304	−1.199	1.536	−0.00166	−3.292	−1.217	1.394	−0.00180	−3.231	−1.222	1.271	−0.00182
0.6	$0.50\phi_k$	−1.118	−0.588	2.633	0.00375	−1.214	−0.567	2.532	0.00387	−1.415	−0.561	2.427	0.00387
	$0.25\phi_k$	2.738	0.084	2.565	0.00361	2.704	0.108	2.663	0.00379	2.597	0.116	2.737	0.00382
	0	8.515	0.372	0.000	−0.01073	8.699	0.380	0.000	−0.01085	9.038	0.381	0.000	−0.01085
	$1.00\phi_k$	−3.509	−1.441	0.000	−0.00412	−2.619	−1.411	0.000	−0.00380	−1.433	−1.323	0.000	−0.00313
	$0.75\phi_k$	−3.133	−1.206	1.314	−0.00158	−2.898	−1.210	0.991	−0.00156	−2.557	−1.195	0.651	−0.00141
1.0	$0.50\phi_k$	−1.509	−0.598	2.432	0.00341	−2.259	−0.641	2.198	0.00304	−3.234	−0.724	1.935	0.00249
	$0.25\phi_k$	2.405	0.080	2.669	0.00349	1.979	0.050	2.857	0.00340	1.381	−0.019	3.043	0.00312
	0	9.452	0.369	0.000	−0.01036	10.809	0.356	0.000	−0.00968	12.587	0.332	0.000	−0.00873
	$1.00\phi_k$	−2.600	−1.378	0.000	−0.00311	−0.737	−1.250	0.000	−0.00223	0.842	−1.115	0.000	−0.00135
	$0.75\phi_k$	−2.808	−1.197	1.035	−0.00130	−2.240	−1.176	0.465	−0.00113	−1.670	−1.146	0.007	−0.00090
1.4	$0.50\phi_k$	−2.222	−0.671	2.190	0.00258	−3.788	−0.800	1.788	0.00176	−5.123	−0.927	1.421	0.00102
	$0.25\phi_k$	1.767	0.013	2.800	0.00305	0.730	−0.096	3.103	0.00268	−0.302	−0.207	3.349	0.00226
	0	11.231	0.344	0.000	−0.00912	14.333	0.306	0.000	−0.00756	17.210	0.269	0.000	−0.00622
	$1.00\phi_k$	−1.608	−1.292	0.000	−0.00208	0.638	−1.117	0.000	−0.00109	1.818	−1.000	0.000	−0.00042
	$0.75\phi_k$	−2.407	−1.176	0.737	−0.00098	−1.594	−1.139	0.056	−0.00076	−0.932	−1.104	−0.322	−0.00056
1.8	$0.50\phi_k$	−2.990	−0.758	1.929	0.00174	−4.907	−0.934	1.424	0.00081	−5.965	−1.047	1.044	0.00023
	$0.25\phi_k$	0.947	−0.079	2.933	0.00250	−0.606	−0.237	3.285	0.00204	−1.895	−0.347	3.497	0.00167
	0	13.499	0.311	0.000	−0.00765	17.937	0.261	0.000	−0.00580	21.202	0.227	0.000	−0.00463
	$1.00\phi_k$	−0.824	−1.213	0.000	−0.00130	1.255	−1.040	0.000	−0.00047	1.839	−0.957	0.000	−0.00005
	$0.75\phi_k$	−2.021	−1.149	0.491	−0.00071	−1.070	−1.109	−0.161	−0.00051	−0.349	−1.073	−0.408	−0.00036
2.2	$0.50\phi_k$	−3.576	−0.825	1.689	0.00110	−5.391	−1.015	1.146	0.00029	−5.987	−1.100	0.768	−0.00009
	$0.25\phi_k$	0.106	−0.162	3.031	0.00201	−1.842	−0.342	3.368	0.00158	−3.411	−0.446	3.523	0.00129
	0	15.827	0.279	0.000	−0.00634	21.146	0.230	0.000	−0.00462	24.769	0.202	0.000	−0.00371
	$1.00\phi_k$	−0.331	−1.153	0.000	−0.00079	1.354	−1.003	0.000	−0.00018	1.434	−0.949	0.000	0.00006
	$0.75\phi_k$	−1.693	−1.122	0.324	−0.00051	−0.663	−1.085	−0.241	−0.00035	0.101	−1.049	−0.375	−0.00023
2.6	$0.50\phi_k$	−3.913	−0.868	1.488	0.00068	−5.417	−1.055	0.934	0.00004	−5.571	−1.118	0.550	−0.00021
	$0.25\phi_k$	−0.674	−0.224	3.083	0.00162	−2.974	−0.418	3.377	0.00126	−4.872	−0.522	3.483	0.00101
	0	18.005	0.251	0.000	−0.00530	24.060	0.208	0.000	−0.00385	28.226	0.184	0.000	−0.00311

$\phi_k = 30$ deg

	$1.00\phi_k$	−3.340	−1.452	0.000	−0.00544	−3.125	−1.482	0.000	−0.00562	−2.714	−1.465	0.000	−0.00540
	$0.75\phi_k$	−2.777	−1.202	1.363	−0.00201	−2.728	−1.216	1.197	−0.00212	−2.621	−1.215	1.045	−0.00208
0.6	$0.50\phi_k$	−0.980	−0.579	2.384	0.00445	−1.167	−0.563	2.256	0.00450	−1.502	−0.581	2.125	0.00431
	$0.25\phi_k$	2.286	0.091	2.392	0.00431	2.194	0.110	2.497	0.00444	2.009	0.097	2.586	0.00437
	0	7.269	0.363	0.000	−0.01263	7.594	0.368	0.000	−0.01268	8.163	0.363	0.000	−0.01238
	$1.00\phi_k$	−2.678	−1.425	0.000	−0.00457	−1.444	−1.336	0.000	−0.00371	−0.148	−1.208	0.000	−0.00263
	$0.75\phi_k$	−2.559	−1.204	1.102	−0.00181	−2.210	−1.193	0.678	−0.00164	−1.807	−1.168	0.287	−0.00137
1.0	$0.50\phi_k$	−1.506	−0.615	2.156	0.00374	−2.531	−0.708	1.845	0.00296	−3.607	−0.826	1.540	0.00207
	$0.25\phi_k$	1.867	0.064	2.513	0.00401	1.248	−0.010	2.750	0.00367	0.538	−0.111	2.960	0.00319
	0	8.463	0.350	0.000	−0.01171	10.345	0.324	0.000	−0.01033	12.402	0.291	0.000	−0.00880
	$1.00\phi_k$	−1.622	−1.325	0.000	−0.00308	0.326	−1.144	0.000	−0.00170	1.453	−1.015	0.000	−0.00074
	$0.75\phi_k$	−2.177	−1.185	0.765	−0.00139	−1.520	−1.147	0.130	−0.00106	−0.981	−1.113	−0.241	−0.00079
1.4	$0.50\phi_k$	−2.345	−0.723	1.876	0.00251	−3.986	−0.896	1.405	0.00130	−4.981	−1.020	1.057	0.00049
	$0.25\phi_k$	1.117	−0.038	2.668	0.00331	−0.089	−0.189	3.007	0.00268	−1.095	−0.303	3.210	0.00219
	0	10.570	0.315	0.000	−0.00975	14.040	0.265	0.000	−0.00747	16.657	0.231	0.000	−0.00594
	$1.00\phi_k$	−0.671	−1.221	0.000	−0.00181	1.168	−1.031	0.000	−0.00062	1.635	−0.947	0.000	−0.00006
	$0.75\phi_k$	−1.763	−1.157	0.454	−0.00098	−0.942	−1.108	−0.171	−0.00067	−0.339	−1.072	−0.386	−0.00046
1.8	$0.50\phi_k$	−3.086	−0.826	1.599	0.00147	−4.684	−1.013	1.074	0.00039	−5.182	−1.100	0.730	−0.00011
	$0.25\phi_k$	0.246	−0.150	2.801	0.00260	−1.366	−0.327	3.137	0.00197	−2.634	−0.428	3.272	0.00160
	0	12.982	0.277	0.000	−0.00782	17.332	0.225	0.000	−0.00559	20.216	0.197	0.000	−0.00446
	$1.00\phi_k$	−0.079	−1.144	0.000	−0.00100	1.285	−0.983	0.000	−0.00016	1.180	−0.940	0.000	0.00012
	$0.75\phi_k$	−1.407	−1.129	0.243	−0.00068	−0.498	−1.078	−0.268	−0.00043	0.155	−1.042	−0.346	−0.00027
2.2	$0.50\phi_k$	−3.521	−0.894	1.368	0.00080	−4.755	−1.068	0.832	−0.00001	−4.762	−1.122	0.481	−0.00029
	$0.25\phi_k$	−0.580	−0.238	2.878	0.00203	−2.544	−0.424	3.162	0.00151	−4.156	−0.521	3.238	0.00121
	0	15.263	0.246	0.000	−0.00634	20.255	0.199	0.000	−0.00447	23.644	0.176	0.000	−0.00362
	$1.00\phi_k$	0.192	−1.094	0.000	−0.00054	1.065	−0.970	0.000	0.00000	0.603	−0.952	0.000	0.00013
	$0.75\phi_k$	−1.127	−1.103	0.128	−0.00046	−0.168	−1.057	−0.261	−0.00028	0.488	−1.020	−0.245	−0.00015
2.6	$0.50\phi_k$	−3.678	−0.931	1.188	0.00042	−4.488	−1.087	0.647	−0.00016	−4.097	−1.119	0.288	−0.00031
	$0.25\phi_k$	−1.320	−0.298	2.900	0.00161	−3.636	−0.494	3.128	0.00118	−5.589	−0.596	3.157	0.00092
	0	17.327	0.221	0.000	−0.00525	23.021	0.181	0.000	−0.00375	27.073	0.161	0.000	−0.00305

TABLE I (CONT.) — INTERNAL FORCES IN A MULTIPLE CYLINDRICAL SHELL DUE TO DEAD LOAD

r/L	φ	r/t = 100				r/t = 200				r/t = 300			
		T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)	T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)	T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)
													ϕ_k = 32.5 deg
0.6	1.00ϕ_k	−2.792	−1.459	0.000	−0.00634	−2.418	−1.463	0.000	−0.00624	−1.831	−1.409	0.000	−0.00562
	0.75ϕ_k	−2.358	−1.203	1.210	−0.00236	−2.264	−1.211	1.013	−0.00239	−2.107	−1.204	0.816	−0.00225
	0.50ϕ_k	−0.894	−0.570	2.168	0.00515	−1.204	−0.576	2.009	0.00500	−1.683	−0.626	1.845	0.00449
	0.25ϕ_k	1.921	0.096	2.247	0.00503	1.754	0.097	2.367	0.00504	1.483	0.057	2.478	0.00480
	0	6.330	0.352	0.000	−0.01460	6.863	0.350	0.000	−0.01434	7.686	0.337	0.000	−0.01352
1.0	1.00ϕ_k	−1.944	−1.391	0.000	−0.00481	−0.480	−1.240	0.000	−0.00327	0.676	−1.096	0.000	−0.00189
	0.75ϕ_k	−2.083	−1.197	0.894	−0.00198	−1.645	−1.170	0.384	−0.00163	−1.231	−1.139	0.003	−0.00127
	0.50ϕ_k	−1.574	−0.646	1.903	0.00390	−2.790	−0.791	1.524	0.00258	−3.771	−0.926	1.212	0.00145
	0.25ϕ_k	1.399	0.034	2.396	0.00443	0.620	−0.087	2.675	0.00377	−0.115	−0.203	2.877	0.00314
	0	7.831	0.327	0.000	−0.01282	10.136	0.287	0.000	−0.01050	12.184	0.251	0.000	−0.00851
1.4	1.00ϕ_k	−0.818	−1.256	0.000	−0.00280	0.887	−1.052	0.000	−0.00109	1.461	−0.953	0.000	−0.00024
	0.75ϕ_k	−1.660	−1.167	0.511	−0.00140	−0.988	−1.118	−0.097	−0.00095	−0.495	−1.083	−0.334	−0.00067
	0.50ϕ_k	−2.475	−0.785	1.590	0.00225	−3.937	−0.979	1.099	0.00077	−4.505	−1.079	0.788	0.00003
	0.25ϕ_k	0.559	−0.101	2.570	0.00345	−0.700	−0.276	2.900	0.00260	−1.669	−0.380	3.037	0.00210
	0	10.187	0.283	0.000	−0.01004	13.647	0.229	0.000	−0.00720	15.930	0.200	0.000	−0.00569
1.8	1.00ϕ_k	−0.028	−1.147	0.000	−0.00142	1.189	−0.975	0.000	−0.00022	1.087	−0.932	0.000	0.00016
	0.75ϕ_k	−1.262	−1.134	0.224	−0.00093	−0.485	−1.079	−0.263	−0.00056	0.070	−1.043	−0.327	−0.00036
	0.50ϕ_k	−3.092	−0.893	1.315	0.00110	−4.197	−1.065	0.807	0.00002	−4.220	−1.120	0.491	−0.00035
	0.25ϕ_k	−0.315	−0.224	2.689	0.00260	−1.918	−0.402	2.966	0.00189	−3.220	−0.496	3.029	0.00151
	0	12.585	0.244	0.000	−0.00777	16.614	0.195	0.000	−0.00539	19.302	0.173	0.000	−0.00435
2.2	1.00ϕ_k	0.328	−1.081	0.000	−0.00068	0.961	−0.957	0.000	0.00004	0.444	−0.948	0.000	0.00018
	0.75ϕ_k	−0.949	−1.106	0.071	−0.00061	−0.110	−1.051	−0.260	−0.00034	0.456	−1.014	−0.210	−0.00018
	0.50ϕ_k	−3.337	−0.953	1.105	0.00049	−3.957	−1.094	0.593	−0.00023	−3.540	−1.119	0.263	−0.00040
	0.25ϕ_k	−1.102	−0.310	2.735	0.00199	−3.070	−0.492	2.943	0.00142	−4.738	−0.588	2.952	0.00110
	0	14.736	0.215	0.000	−0.00622	19.385	0.174	0.000	−0.00435	22.713	0.155	0.000	−0.00354
2.6	1.00ϕ_k	0.402	−1.047	0.000	−0.00032	0.593	−0.961	0.000	0.00010	−0.072	−0.970	0.000	0.00012
	0.75ϕ_k	−0.719	−1.082	0.011	−0.00041	0.141	−1.031	−0.197	−0.00020	0.637	−0.996	−0.089	−0.00008
	0.50ϕ_k	−3.325	−0.978	0.947	0.00019	−3.510	−1.096	0.430	−0.00028	−2.773	−1.102	0.104	−0.00035
	0.25ϕ_k	−1.796	−0.366	2.727	0.00156	−4.124	−0.559	2.876	0.00109	−6.079	−0.663	2.840	0.00081
	0	16.677	0.194	0.000	−0.00514	22.090	0.159	0.000	−0.00366	26.101	0.142	0.000	−0.00298

$\phi_k = 35$ deg

	$1.00\phi_k$	−2.323	−1.458	0.000	−0.00718	−1.769	−1.422	0.000	−0.00658	−1.037	−1.329	0.000	−0.00542
	$0.75\phi_k$	−2.015	−1.202	1.072	−0.00270	−1.870	−1.201	0.831	−0.00260	−1.668	−1.186	0.583	−0.00233
0.6	$0.50\phi_k$	−0.852	−0.567	1.976	0.00581	−1.307	−0.607	1.781	0.00527	−1.906	−0.692	1.582	0.00433
	$0.25\phi_k$	1.615	0.096	2.127	0.00573	1.361	0.067	2.268	0.00553	1.011	−0.001	2.404	0.00505
	0	5.630	0.338	0.000	−0.01654	6.414	0.327	0.000	−0.01566	7.466	0.305	0.000	−0.01413
	$1.00\phi_k$	−1.293	−1.339	0.000	−0.00476	0.207	−1.141	0.000	−0.00256	1.047	−1.008	0.000	−0.00111
	$0.75\phi_k$	−1.681	−1.184	0.689	−0.00208	−1.193	−1.144	0.139	−0.00153	−0.805	−1.110	−0.172	−0.00113
1.0	$0.50\phi_k$	−1.683	−0.691	1.668	0.00383	−2.941	−0.877	1.248	0.00199	−3.686	−1.005	0.959	0.00078
	$0.25\phi_k$	0.983	−0.010	2.309	0.00473	0.111	−0.169	2.608	0.00374	−0.593	−0.283	2.773	0.00303
	0	7.451	0.300	0.000	−0.01356	9.963	0.250	0.000	−0.01028	11.807	0.216	0.000	−0.00810
	$1.00\phi_k$	−0.215	−1.181	0.000	−0.00233	1.037	−0.987	0.000	−0.00053	1.124	−0.927	0.000	0.00011
	$0.75\phi_k$	−1.244	−1.145	0.291	−0.00134	−0.602	−1.090	−0.211	−0.00083	−0.145	−1.054	−0.316	−0.00055
1.4	$0.50\phi_k$	−2.553	−0.851	1.337	0.00183	−3.666	−1.038	0.863	0.00027	−3.847	−1.109	0.583	−0.00031
	$0.25\phi_k$	0.096	−0.170	2.490	0.00347	−1.147	−0.348	2.771	0.00250	−2.121	−0.442	2.844	0.00201
	0	9.917	0.250	0.000	−0.01001	13.132	0.199	0.000	−0.00689	15.196	0.175	0.000	−0.00551
	$1.00\phi_k$	0.340	−1.081	0.000	−0.00098	0.920	−0.948	0.000	0.00005	0.463	−0.940	0.000	0.00025
	$0.75\phi_k$	−0.881	−1.110	0.060	−0.00084	−0.162	−1.053	−0.255	−0.00046	0.335	−1.016	−0.206	−0.00026
1.8	$0.50\phi_k$	−2.982	−0.952	1.079	0.00070	−3.570	−1.091	0.602	−0.00026	−3.246	−1.118	0.305	−0.00049
	$0.25\phi_k$	−0.748	−0.294	2.580	0.00254	−2.342	−0.464	2.781	0.00179	−3.701	−0.556	2.785	0.00140
	0	12.191	0.213	0.000	−0.00758	15.897	0.171	0.000	−0.00523	18.537	0.152	0.000	−0.00425
	$1.00\phi_k$	0.474	−1.033	0.000	−0.00039	0.514	−0.953	0.000	0.00015	−0.138	−0.969	0.000	0.00016
	$0.75\phi_k$	−0.615	−1.083	−0.028	−0.00053	0.141	−1.026	−0.185	−0.00025	0.574	−0.991	−0.062	−0.00010
2.2	$0.50\phi_k$	−3.041	−0.996	0.893	0.00019	−3.129	−1.098	0.408	−0.00037	−2.446	−1.099	0.110	−0.00044
	$0.25\phi_k$	−1.491	−0.374	2.593	0.00192	−3.474	−0.551	2.722	0.00132	−5.149	−0.649	2.674	0.00098
	0	14.200	0.189	0.000	−0.00605	18.611	0.153	0.000	−0.00424	21.916	0.136	0.000	−0.00345
	$1.00\phi_k$	0.405	−1.014	0.000	−0.00014	0.141	−0.968	0.000	0.00012	−0.465	−0.990	0.000	0.00008
	$0.75\phi_k$	−0.431	−1.062	−0.040	−0.00034	0.301	−1.009	−0.096	−0.00013	0.588	−0.980	0.047	−0.00002
2.6	$0.50\phi_k$	−2.899	−1.010	0.757	−0.00001	−2.596	−1.088	0.272	−0.00035	−1.678	−1.075	−0.002	−0.00034
	$0.25\phi_k$	−2.142	−0.425	2.559	0.00149	−4.465	−0.617	2.629	0.00098	−6.332	−0.720	2.537	0.00068
	0	16.045	0.170	0.000	−0.00501	21.274	0.140	0.000	−0.00357	25.213	0.125	0.000	−0.00290

TABLE I (CONT.) — INTERNAL FORCES IN A MULTIPLE CYLINDRICAL SHELL DUE TO DEAD LOAD

r/L	φ	r/t = 100				r/t = 200				r/t = 300			
		T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)	T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)	T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)
													$\phi_k = 37.5$ deg
0.6	$1.00\phi_k$	−1.905	−1.446	0.000	−0.00787	−1.175	−1.361	0.000	−0.00654	−0.375	−1.234	0.000	0.00480
	$0.75\phi_k$	−1.728	−1.199	0.944	−0.00300	−1.531	−1.187	0.647	−0.00271	−1.298	−1.165	0.360	0.00230
	$0.50\phi_k$	−0.849	−0.573	1.803	0.00636	−1.448	−0.656	1.567	0.00524	−2.110	−0.771	1.343	0.00383
	$0.25\phi_k$	1.349	0.088	2.028	0.00638	1.005	0.023	2.197	0.00586	0.601	−0.071	2.351	0.00513
	0	5.119	0.321	0.000	−0.01834	6.165	0.299	0.000	−0.01651	7.632	0.270	0.000	−0.01419
1.0	$1.00\phi_k$	−0.738	−1.273	0.000	−0.00441	0.602	−1.056	0.000	−0.00176	1.072	−0.951	0.000	−0.00044
	$0.75\phi_k$	−1.341	−1.167	0.494	−0.00209	−0.847	−1.118	−0.034	−0.00139	−0.491	−1.083	−0.245	−0.00098
	$0.50\phi_k$	−1.799	−0.747	1.451	0.00353	−2.941	−0.952	1.023	0.00130	−3.405	−1.058	0.768	0.00019
	$0.25\phi_k$	0.616	−0.065	2.244	0.00488	−0.277	−0.243	2.532	0.00363	−0.948	−0.347	2.646	0.00292
	0	7.225	0.270	0.000	−0.01387	9.713	0.217	0.000	−0.00985	11.317	0.188	0.000	−0.00773
1.4	$1.00\phi_k$	0.180	−1.109	0.000	−0.00176	0.915	−0.949	0.000	−0.00010	0.653	−0.926	0.000	0.00030
	$0.75\phi_k$	−0.916	−1.122	0.122	−0.00124	−0.319	−1.065	−0.235	−0.00070	0.106	−1.028	−0.232	−0.00043
	$0.50\phi_k$	−2.544	−0.913	1.122	0.00132	−3.251	−1.073	0.682	−0.00014	−3.131	−1.115	0.421	−0.00055
	$0.25\phi_k$	−0.272	−0.238	2.412	0.00340	−1.488	−0.407	2.626	0.00240	−2.505	−0.496	2.645	0.00190
	0	9.655	0.219	0.000	−0.00976	12.569	0.174	0.000	−0.00664	14.553	0.154	0.000	−0.00537
1.8	$1.00\phi_k$	0.482	−1.029	0.000	−0.00058	0.536	−0.944	0.000	0.00020	−0.061	−0.960	0.000	0.00024
	$0.75\phi_k$	−0.597	−1.087	−0.036	−0.00074	0.061	−1.029	−0.187	−0.00035	0.465	−0.992	−0.089	−0.00015
	$0.50\phi_k$	−2.768	−0.996	0.890	0.00032	−2.902	−1.097	0.442	−0.00045	−2.353	−1.101	0.167	−0.00055
	$0.25\phi_k$	−1.074	−0.355	2.467	0.00244	−2.681	−0.518	2.592	0.00168	−4.064	−0.612	2.548	0.00126
	0	11.765	0.187	0.000	−0.00733	15.251	0.151	0.000	−0.00509	17.883	0.134	0.000	−0.00414
2.2	$1.00\phi_k$	0.444	−1.001	0.000	−0.00016	0.098	−0.962	0.000	0.00017	−0.480	−0.990	0.000	0.00011
	$0.75\phi_k$	−0.375	−1.062	−0.066	−0.00045	0.277	−1.005	−0.083	−0.00016	0.532	−0.975	0.066	−0.00002
	$0.50\phi_k$	−2.675	−1.024	0.726	−0.00005	−2.353	−1.088	0.271	−0.00044	−1.531	−1.072	0.019	−0.00043
	$0.25\phi_k$	−1.779	−0.429	2.449	0.00182	−3.770	−0.604	2.505	0.00119	−5.375	−0.701	2.410	0.00083
	0	13.662	0.166	0.000	−0.00587	17.929	0.135	0.000	−0.00413	21.172	0.120	0.000	−0.00335
2.6	$1.00\phi_k$	0.294	−0.994	0.000	−0.00002	−0.192	−0.981	0.000	0.00011	−0.598	−1.005	0.000	0.00004
	$0.75\phi_k$	−0.234	−1.043	−0.043	−0.00027	0.338	−0.992	0.007	−0.00007	0.404	−0.971	0.145	0.00003
	$0.50\phi_k$	−2.446	−1.027	0.609	−0.00016	−1.802	−1.070	0.166	−0.00036	−0.825	−1.043	−0.040	−0.00031
	$0.25\phi_k$	−2.390	−0.477	2.394	0.00139	−4.662	−0.667	2.393	0.00085	−6.368	−0.767	2.252	0.00055
	0	15.442	0.150	0.000	−0.00488	20.531	0.124	0.000	−0.00347	24.353	0.110	0.000	−0.00281

$\phi_k = 40$ deg

	$1.00\phi_k$	−1.524	−1.422	0.000	−0.00834	−0.655	−1.284	0.000	−0.00609	0.113	−1.139	0.000	−0.00386
	$0.75\phi_k$	−1.481	−1.192	0.819	−0.00325	−1.239	−1.169	0.466	−0.00273	−0.994	−1.142	0.169	−0.00218
0.6	$0.50\phi_k$	−0.879	−0.590	1.644	0.00674	−1.595	−0.718	1.389	0.00488	−2.241	−0.851	1.137	0.00307
	$0.25\phi_k$	1.112	0.070	1.949	0.00694	0.689	−0.033	2.145	0.00602	0.264	−0.143	2.301	0.00506
	0	4.760	0.300	0.000	−0.01987	6.034	0.268	0.000	−0.01681	7.260	0.235	0.000	−0.01379
	$1.00\phi_k$	−0.296	−1.201	0.000	−0.00381	0.744	−0.992	0.000	−0.00099	0.882	−0.924	0.000	0.00005
	$0.75\phi_k$	−1.058	−1.146	0.319	−0.00202	−0.586	−1.093	−0.131	−0.00124	−0.254	−1.058	−0.243	−0.00084
1.0	$0.50\phi_k$	−1.885	−0.808	1.256	0.00301	−2.803	−1.008	0.848	0.00064	−3.007	−1.087	0.623	−0.00028
	$0.25\phi_k$	0.304	−0.125	2.191	0.00490	−0.568	−0.306	2.440	0.00349	−1.231	−0.399	2.503	0.00281
	0	7.066	0.239	0.000	−0.01379	9.372	0.188	0.000	−0.00939	10.804	0.164	0.000	−0.00744
	$1.00\phi_k$	0.385	−1.049	0.000	−0.00118	0.649	−0.935	0.000	0.00018	0.191	−0.942	0.000	0.00036
	$0.75\phi_k$	−0.665	−1.098	0.009	−0.00112	−0.111	−1.041	−0.198	−0.00058	0.269	−1.004	−0.119	−0.00031
1.4	$0.50\phi_k$	−2.444	−0.963	0.947	0.00080	−2.770	−1.087	0.539	−0.00044	−2.437	−1.105	0.292	−0.00068
	$0.25\phi_k$	−0.555	−0.299	2.329	0.00329	−1.766	−0.456	2.473	0.00228	−2.824	−0.546	2.450	0.00176
	0	9.355	0.191	0.000	−0.00941	12.033	0.153	0.000	−0.00643	14.007	0.136	0.000	−0.00523
	$1.00\phi_k$	0.467	−0.994	0.000	−0.00026	0.159	−0.953	0.000	0.00025	−0.404	−0.983	0.000	0.00018
	$0.75\phi_k$	−0.387	−1.065	−0.075	−0.00063	0.198	−1.007	−0.092	−0.00025	0.469	−0.975	0.054	−0.00006
1.8	$0.50\phi_k$	−2.483	−1.024	0.739	0.00000	−2.262	−1.088	0.318	−0.00056	−1.586	−1.074	0.079	−0.00056
	$0.25\phi_k$	−1.320	−0.408	2.348	0.00233	−2.946	−0.566	2.405	0.00154	−4.293	−0.661	2.322	0.00110
	0	11.320	0.164	0.000	−0.00708	14.681	0.133	0.000	−0.00495	17.274	0.118	0.000	−0.00401
	$1.00\phi_k$	0.319	−0.983	0.000	−0.00000	−0.210	−0.977	0.000	0.00015	−0.599	−1.006	0.000	0.00005
	$0.75\phi_k$	−0.206	−1.042	−0.060	−0.00036	0.313	−0.988	0.019	−0.00009	0.376	−0.966	0.159	0.00003
2.2	$0.50\phi_k$	−2.281	−1.038	0.594	−0.00023	−1.673	−1.068	0.178	−0.00046	−0.808	−1.040	−0.012	−0.00039
	$0.25\phi_k$	−1.992	−0.477	2.306	0.00171	−3.953	−0.650	2.296	0.00105	−5.427	−0.745	2.160	0.00069
	0	13.143	0.146	0.000	−0.00569	17.303	0.119	0.000	−0.00401	20.442	0.106	0.000	−0.00324
	$1.00\phi_k$	0.140	−0.986	0.000	0.00005	−0.380	−0.994	0.000	0.00007	−0.558	−1.013	0.000	0.00000
	$0.75\phi_k$	−0.107	−1.026	−0.016	−0.00021	0.281	−0.980	0.094	−0.00001	0.164	−0.969	0.204	0.00005
2.6	$0.50\phi_k$	−2.003	−1.032	0.495	−0.00025	−1.150	−1.045	0.108	−0.00035	−0.202	−1.011	−0.023	−0.00026
	$0.25\phi_k$	−2.558	−0.523	2.236	0.00128	−4.723	−0.709	2.170	0.00072	−6.226	−0.804	1.985	0.00043
	0	14.875	0.132	0.000	−0.00474	19.822	0.109	0.000	−0.00336	23.509	0.097	0.000	−0.00272

TABLE I (CONT.) — INTERNAL FORCES IN A MULTIPLE CYLINDRICAL SHELL DUE TO DEAD LOAD

r/L	φ	r/t = 100				r/t = 200				r/t = 300			
		T_ϕ (1)	T_θ (2)	S (3)	M_θ (4)	T_ϕ (1)	T_θ (2)	S (3)	M_θ (4)	T_ϕ (1)	T_θ (2)	S (3)	M_θ (4)
													$\phi_k = 45$ deg
	1.00ϕ_k	−1.416	−1.446	0.000	−0.01148	−0.959	−1.368	0.000	−0.00974	−0.439	−1.249	0.000	−0.00734
	0.75ϕ_k	−1.244	−1.185	0.832	−0.00425	−1.125	−1.174	0.619	−0.00387	−0.980	−1.155	0.400	−0.00331
0.4	0.50ϕ_k	−0.540	−0.546	1.556	0.00930	−0.915	−0.618	1.380	0.00787	−1.344	−0.726	1.212	0.00593
	0.25ϕ_k	1.004	0.094	1.706	0.00889	0.797	0.038	1.829	0.00822	0.547	−0.046	1.947	0.00725
	0	3.456	0.278	0.000	−0.02554	4.094	0.261	0.000	−0.02328	4.847	0.237	0.000	−0.02017
	1.00ϕ_k	−0.856	−1.335	0.000	−0.00834	0.070	−1.121	0.000	−0.00423	0.545	−0.993	0.000	−0.00175
	0.75ϕ_k	−1.078	−1.168	0.576	−0.00350	−0.789	−1.127	0.165	−0.00249	−0.567	−1.095	−0.059	−0.00181
0.6	0.50ϕ_k	−0.999	−0.654	1.356	0.00674	−1.777	−0.852	1.038	0.00337	−2.207	−0.977	0.839	0.00130
	0.25ϕ_k	0.701	0.007	1.837	0.00765	0.194	−0.156	2.061	0.00588	−0.197	−0.263	2.175	0.00473
	0	4.357	0.251	0.000	−0.02168	5.836	0.205	0.000	−0.01601	6.864	0.175	0.000	−0.01245
	1.00ϕ_k	0.213	−1.068	0.000	−0.00224	0.559	−0.932	0.000	0.00009	0.273	−0.929	0.000	0.00051
	0.75ϕ_k	−0.644	−1.103	0.075	−0.00174	−0.235	−1.048	−0.149	−0.00093	0.059	−1.011	−0.109	−0.00055
1.0	0.50ϕ_k	−1.892	−0.918	0.949	0.00166	−2.271	−1.062	0.608	−0.00037	−2.110	−1.090	0.410	−0.00084
	0.25ϕ_k	−0.152	−0.240	2.063	0.00465	−0.973	−0.398	2.220	0.00321	−1.686	−0.482	2.210	0.00254
	0	6.719	0.182	0.000	−0.01287	8.595	0.144	0.000	−0.00862	9.920	0.128	0.000	−0.00699
	1.00ϕ_k	0.396	−0.976	0.000	−0.00025	0.049	−0.951	0.000	0.00037	−0.413	−0.985	0.000	0.00024
	0.75ϕ_k	−0.329	−1.056	−0.067	−0.00085	0.131	−0.999	−0.040	−0.00033	0.338	−0.967	0.098	−0.00007
1.4	0.50ϕ_k	−2.047	−1.021	0.694	−0.00007	−1.812	−1.073	0.336	−0.00075	−1.270	−1.057	0.138	−0.00073
	0.25ϕ_k	−0.937	−0.393	2.142	0.00300	−2.180	−0.539	2.168	0.00197	−3.215	−0.632	2.085	0.00141
	0	8.668	0.146	0.000	−0.00868	11.123	0.119	0.000	−0.00604	13.049	0.105	0.000	−0.00489
	1.00ϕ_k	0.209	−0.966	0.000	0.00013	−0.341	−0.985	0.000	0.00018	−0.576	−1.013	0.000	0.00003
	0.75ϕ_k	−0.126	−1.026	−0.040	−0.00043	0.246	−0.974	0.094	−0.00006	0.214	−0.959	0.213	0.00007
1.8	0.50ϕ_k	−1.833	−1.041	0.525	−0.00043	−1.194	−1.045	0.174	−0.00060	−0.470	−1.011	0.040	−0.00047
	0.25ϕ_k	−1.650	−0.490	2.106	0.00205	−3.232	−0.645	2.053	0.00121	−4.371	−0.735	1.907	0.00076
	0	10.464	0.127	0.000	−0.00661	13.662	0.103	0.000	−0.00462	16.071	0.091	0.000	−0.00373
	1.00ϕ_k	0.001	−0.977	0.000	0.00013	−0.457	−1.004	0.000	0.00005	−0.463	−1.017	0.000	−0.00002
	0.75ϕ_k	−0.027	−1.008	0.021	−0.00021	0.169	−0.968	0.171	0.00003	−0.042	−0.966	0.246	0.00007
2.2	0.50ϕ_k	−1.526	−1.033	0.415	−0.00042	−0.657	−1.015	0.111	−0.00041	0.115	−0.975	0.050	−0.00027
	0.25ϕ_k	−2.245	−0.554	2.030	0.00144	−4.003	−0.719	1.912	0.00075	−5.134	−0.803	1.710	0.00041
	0	12.191	0.113	0.000	−0.00533	16.103	0.092	0.000	−0.00373	19.003	0.082	0.000	−0.00301

$\phi_L = 50$ deg

	$1.00\phi_L$	−0.998	−1.399	0.000	−0.01249	−0.404	−1.239	0.000	−0.00849	0.064	−1.097	0.000	−0.00500
	$0.75\phi_L$	−0.976	−1.168	0.678	−0.00474	−0.813	−1.142	0.393	−0.00382	−0.661	−1.116	0.166	−0.00298
0.4	$0.50\phi_L$	−0.575	−0.570	1.352	0.01015	−1.066	−0.716	1.132	0.00690	−1.466	−0.847	0.961	0.00405
	$0.25\phi_L$	0.747	0.065	1.598	0.01001	0.465	−0.050	1.757	0.00840	0.202	−0.155	1.876	0.00695
	0	3.061	0.237	0.000	−0.02868	3.922	0.206	0.000	−0.02344	4.682	0.178	0.000	−0.01879
	$1.00\phi_L$	−0.342	−1.214	0.000	−0.00700	0.341	−1.000	0.000	−0.00204	0.468	−0.928	0.000	−0.00015
	$0.75\phi_L$	−0.772	−1.135	0.354	−0.00338	−0.497	−1.085	0.005	−0.00207	−0.306	−1.052	−0.098	−0.00142
0.6	$0.50\phi_L$	−1.119	−0.747	1.111	0.00565	−1.720	−0.951	0.815	0.00153	−1.885	−1.034	0.661	−0.00012
	$0.25\phi_L$	0.374	−0.062	1.766	0.00771	−0.115	−0.256	1.955	0.00545	−0.474	−0.343	2.006	0.00437
	0	4.162	0.197	0.000	−0.02153	5.499	0.152	0.000	−0.01441	6.297	0.132	0.000	−0.01125
	$1.00\phi_L$	0.307	−0.984	0.000	−0.00080	0.152	−0.936	0.000	0.00053	−0.231	−0.967	0.000	0.00046
	$0.75\phi_L$	−0.388	−1.063	−0.016	−0.00139	−0.031	−1.007	−0.041	−0.00062	0.181	−0.973	0.069	−0.00024
1.0	$0.50\phi_L$	−1.684	−0.983	0.745	0.00039	−1.644	−1.058	0.453	−0.00091	−1.304	−1.051	0.278	−0.00101
	$0.25\phi_L$	−0.428	−0.326	1.951	0.00426	−1.259	−0.464	1.990	0.00288	−1.990	−0.552	1.937	0.00214
	0	6.251	0.137	0.000	−0.01169	7.902	0.110	0.000	−0.00802	9.207	0.098	0.000	−0.00650
	$1.007\phi_L$	0.159	−0.956	0.000	0.00023	−0.334	−0.984	0.000	0.00026	−0.534	−1.014	0.000	0.00005
	$0.75\phi_L$	−0.139	−1.020	−0.017	−.000059	0.168	−0.968	0.123	−0.00009	0.149	−0.953	0.231	0.00009
1.4	$0.50\phi_L$	−1.549	−1.029	0.533	−0.00059	−1.032	−1.028	0.232	−0.00079	−0.467	−0.995	0.118	−0.00062
	$0.25\phi_L$	−1.171	−0.460	1.943	0.00256	−2.385	−0.604	1.886	0.00158	−3.262	−0.691	1.760	0.00101
	0	7.993	0.112	0.000	−0.00802	10.330	0.091	0.000	−0.00559	12.097	0.080	0.000	−0.00449
	$1.00\phi_L$	−0.084	−0.973	0.000	0.00022	−0.458	−1.009	0.000	0.00005	−0.406	−1.018	0.000	−0.00003
	$0.75\phi_L$	−0.016	−0.995	0.060	−0.00023	0.088	−0.960	0.214	0.00006	−0.126	−0.963	0.269	0.00010
1.8	$0.50\phi_L$	−1.228	−1.022	0.398	−0.00060	−0.465	−0.989	0.154	−0.00050	0.148	−0.949	0.127	−0.00031
	$0.25\phi_L$	−1.819	−0.551	1.872	0.00172	−3.209	−0.698	1.738	0.00086	−4.075	−0.775	1.547	0.00045
	0	9.694	0.097	0.000	−0.00612	12.669	0.079	0.000	−0.00425	14.873	0.070	0.000	−0.00343
	$1.00\phi_L$	−0.223	−0.989	0.000	0.00011	−0.398	−1.013	0.000	−0.00001	−0.263	−1.012	0.000	−0.00002
	$0.75\phi_L$	0.000	−0.984	0.123	−0.00008	−0.076	−0.965	0.243	0.00007	−0.321	−0.975	0.259	0.00005
2.2	$0.50\phi_L$	−0.919	−1.002	0.328	−0.00045	−0.066	−0.958	0.160	−0.00029	0.463	−0.920	0.193	−0.00013
	$0.25\phi_L$	−2.303	−0.609	1.777	0.00113	−3.727	−0.758	1.578	0.00046	−4.492	−0.826	1.339	0.00017
	0	11.317	0.087	0.000	0.00492	14.917	0.070	0.000	−0.00343	17.586	0.062	0.000	−0.00277

TABLE I (CONT.) — INTERNAL FORCES IN A MULTIPLE CYLINDRICAL SHELL DUE TO DEAD LOAD

r/L	φ	r/t = 100				r/t = 200				r/t = 300			
		T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)	T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)	T_x (1)	T_ϕ (2)	S (3)	M_ϕ (4)
													$\phi_k = 55$ deg
	$1.00\phi_k$	—0.640	—1.318	0.000	—0.01218	—0.048	—1.109	0.000	—0.00603	0.243	—0.991	0.000	—0.00246
	$0.75\phi_k$	—0.764	—1.143	0.528	—0.00490	—0.585	—1.106	0.215	—0.00344	—0.449	—1.076	0.045	—0.00250
0.4	$0.50\phi_k$	—0.642	—0.622	1.174	0.00995	—1.143	—0.815	0.939	0.00492	—1.411	—0.932	0.796	0.00194
	$0.25\phi_k$	0.532	0.011	1.525	0.01053	0.221	—0.142	1.697	0.00799	—0.013	—0.240	1.784	0.00639
	0	2.847	0.191	0.000	—0.02996	3.772	0.154	0.000	—0.02174	4.395	0.130	0.000	—0.01672
	$1.00\phi_k$	—0.032	—1.097	0.000	—0.00479	0.298	—0.941	0.000	—0.00037	0.192	—0.924	0.000	0.00066
	$0.75\phi_k$	—0.552	—1.097	0.193	—0.00298	—0.313	—1.047	—0.023	—0.00165	—0.140	—1.013	—0.026	—0.00105
0.6	$0.50\phi_k$	—1.157	—0.836	0.922	0.00383	—1.488	—0.997	0.679	0.00005	—1.463	—1.037	0.547	—0.00098
	$0.25\phi_k$	0.140	—0.170	1.701	0.00728	—0.304	—0.322	1.820	0.00500	—0.671	—0.398	1.826	0.00400
	0	3.977	0.147	0.000	—0.01988	5.054	0.113	0.000	—0.01293	5.755	0.100	0.000	—0.01033
	$1.00\phi_k$	0.179	—0.951	0.000	0.00011	—0.189	—0.966	0.000	0.00051	—0.448	—1.002	0.000	0.00022
	$0.75\phi_k$	—0.231	—1.028	0.003	—0.00105	0.055	—0.974	0.097	—0.00031	0.131	—0.951	0.205	0.00001
1.0	$0.50\phi_k$	—1.369	—1.001	0.616	—0.00048	—1.086	—1.021	0.359	—0.00108	—0.697	—0.995	0.232	—0.00094
	$0.25\phi_k$	—0.599	—0.384	1.801	0.00385	—1.439	—0.517	1.775	0.00244	—2.098	—0.602	1.693	0.00166
	0	5.748	0.103	0.000	—0.01063	7.308	0.083	0.000	—0.00737	8.519	0.073	0.000	—0.00592
	$1.00\phi_k$	—0.092	—0.966	0.000	0.00035	—0.437	—1.008	0.000	0.00009	—0.407	—1.019	0.000	—0.00004
	$0.75\phi_k$	—0.049	—0.990	0.081	—0.00035	0.057	—0.953	0.230	0.00007	—0.108	—0.955	0.284	0.00013
1.4	$0.50\phi_k$	—1.083	—1.006	0.435	—0.00081	—0.483	—0.973	0.220	—0.00068	0.006	—0.934	0.190	—0.00044
	$0.25\phi_k$	—1.301	—0.509	1.751	0.00226	—2.378	—0.645	1.635	0.00116	—3.061	—0.721	1.479	0.00065
	0	7.382	0.084	0.000	—0.00735	9.540	0.068	0.000	—0.00508	11.136	0.060	0.000	—0.00408
	$1.00\phi_k$	—0.264	—0.989	0.000	0.00016	—0.376	—1.015	0.000	—0.00002	—0.246	—1.012	0.000	—0.00003
	$0.75\phi_k$	—0.013	—0.973	0.159	—0.00008	—0.119	—0.959	0.268	0.00009	—0.328	—0.971	0.274	0.00006
1.8	$0.50\phi_k$	—0.752	—0.983	0.342	—0.00060	—0.043	—0.933	0.218	—0.00035	0.368	—0.897	0.260	—0.00015
	$0.25\phi_k$	—1.846	—0.592	1.658	0.00135	—2.964	—0.723	1.467	0.00054	—3.552	—0.785	1.255	0.00020
	0	8.970	0.073	0.000	—0.00558	11.697	0.058	0.000	—0.00397	13.680	0.052	0.000	—0.00311
	$1.00\phi_k$	—0.310	—1.001	0.000	0.00005	—0.285	—1.012	0.000	—0.00002	—0.200	—1.007	0.000	—0.00001
	$0.75\phi_k$	—0.059	—0.969	0.202	0.00001	—0.253	—0.968	0.265	0.00005	—0.380	—0.979	0.250	0.00001
2.2	$0.50\phi_k$	—0.495	—0.959	0.313	—0.00039	0.186	—0.908	0.264	—0.00016	0.426	—0.880	0.340	—0.00003
	$0.25\phi_k$	—2.201	—0.642	1.553	0.00082	—3.252	—0.768	1.302	0.00023	—3.664	—0.819	1.057	0.00000
	0	10.467	0.065	0.000	—0.00447	13.734	0.053	0.000	—0.00311	16.157	0.047	0.000	—0.00251

ϕ_k = 60 deg

	$1.00\phi_k$	—0.358	—1.218	0.000	—0.01047	0.099	—1.012	0.000	—0.00336	0.205	—0.941	0.000	—0.00054
	$0.75\phi_k$	—0.597	—1.113	0.392	—0.00469	—0.428	—1.070	0.117	—0.00291	—0.312	—1.040	0.026	—0.00202
0.4	$0.50\phi_k$	—0.705	—0.691	1.022	0.00862	—1.107	—0.886	0.808	0.00272	—1.234	—0.968	0.697	0.00026
	$0.25\phi_k$	0.358	—0.056	1.474	0.01038	0.064	—0.215	1.624	0.00731	—0.146	—0.294	1.670	0.00584
	0	2.715	0.144	0.000	—0.02912	3.546	0.110	0.000	—0.01928	4.032	0.094	0.000	—0.01484
	$1.00\phi_k$	0.083	—1.012	0.000	—0.00256	0.120	—0.932	0.000	—0.00056	—0.086	—0.949	0.000	0.00083
	$0.75\phi_k$	—0.404	—1.062	0.113	—0.00249	—0.194	—1.012	0.029	—0.00125	—0.047	—0.979	0.085	—0.00067
0.6	$0.50\phi_k$	—1.092	—0.896	0.793	0.00195	—1.192	—0.998	0.594	—0.00088	—1.057	—1.007	0.471	—0.00135
	$0.25\phi_k$	—0.009	—0.238	1.622	0.00664	—0.432	—0.364	1.677	0.00453	—0.815	—0.437	1.657	0.00351
	0	3.723	0.106	0.000	—0.01769	4.617	0.083	0.000	—0.01170	5.291	0.073	0.000	—0.00943
	$1.00\phi_k$	—0.010	—0.952	0.000	0.00050	—0.360	—0.995	0.000	0.00030	—0.440	—1.018	0.000	0.00002
	$0.75\phi_k$	—0.138	—0.998	0.076	—0.00073	0.036	—0.952	0.208	—0.00005	—0.016	—0.944	0.280	0.00015
1.0	$0.50\phi_k$	—1.042	—0.986	0.535	—0.00095	—0.656	—0.970	0.325	—0.00102	—0.290	—0.936	0.259	—0.00075
	$0.25\phi_k$	—0.708	—0.424	1.651	0.00338	—1.494	—0.553	1.583	0.00193	—2.033	—0.629	1.481	0.00120
	0	5.279	0.075	0.000	—0.00964	6.733	0.061	0.000	—0.00664	7.809	0.054	0.000	—0.00530
	$1.00\phi_k$	—0.255	—0.985	0.000	0.00027	—0.381	—1.016	0.000	—0.00002	—0.270	—1.014	0.000	—0.00005
	$0.75\phi_k$	—0.038	—0.967	0.176	—0.00014	—0.103	—0.951	0.284	0.00012	—0.284	—0.963	0.295	0.00010
1.4	$0.50\phi_k$	—0.708	—0.965	0.391	—0.00082	—0.144	—0.918	0.274	—0.00050	0.210	—0.881	0.301	—0.00025
	$0.25\phi_k$	—1.333	—0.541	1.575	0.00181	—2.213	—0.662	1.418	0.00079	—2.703	—0.725	1.249	0.00034
	0	6.804	0.062	0.000	—0.00662	8.739	0.050	0.000	—0.00455	10.176	0.044	0.000	—0.00365
	$1.00\phi_k$	—0.323	—1.003	0.000	0.00007	—0.276	—1.012	0.000	—0.00003	—0.202	—1.006	0.000	—0.00001
	$0.75\phi_k$	—0.074	—0.960	0.232	0.00002	—0.261	—0.963	0.285	0.00007	—0.364	—0.974	0.268	0.00002
1.8	$0.50\phi_k$	—0.420	—0.937	0.344	—0.00051	0.136	—0.884	0.318	—0.00020	0.322	—0.858	0.389	—0.00004
	$0.25\phi_k$	—1.753	—0.613	1.470	0.00099	—2.579	—0.722	1.244	0.00028	—2.906	—0.770	1.035	0.00001
	0	8.252	0.053	0.000	—0.00500	10.671	0.043	0.000	—0.00345	12.469	0.038	0.000	—0.00277
	$1.00\phi_k$	—0.309	—1.008	0.000	0.00001	—0.231	—1.008	0.000	—0.00001	—0.222	—1.005	0.000	0.00000
	$0.75\phi_k$	—0.143	—0.962	0.251	0.00004	—0.315	—0.971	0.269	0.00002	—0.324	—0.976	0.248	—0.00001
2.2	$0.50\phi_k$	—0.234	—0.915	0.347	—0.00029	0.207	—0.869	0.375	—0.00006	0.216	—0.854	0.455	0.00003
	$0.25\phi_k$	—1.986	—0.653	1.361	0.00055	—2.675	—0.753	1.090	0.00005	—2.787	—0.787	0.862	—0.00011
	0	9.616	0.047	0.000	—0.00399	12.536	0.038	0.000	—0.00277	14.699	0.034	0.000	—0.00223

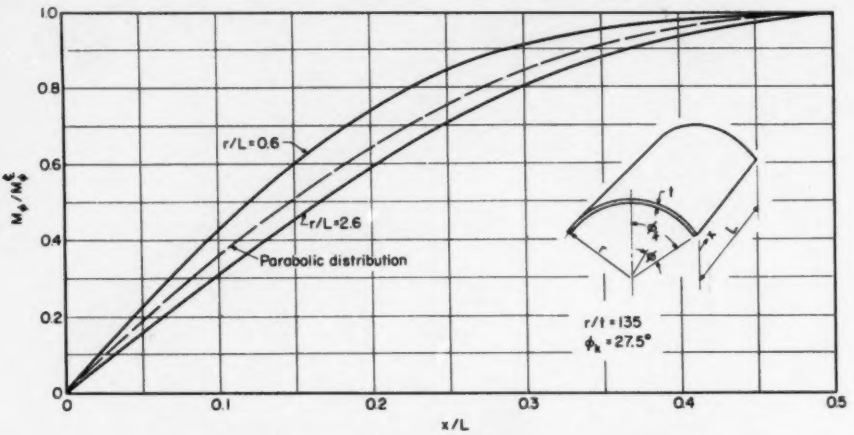


Fig. 5—Longitudinal distribution of transverse moment at crown

point, and there remains essentially uniform to midspan. On the other hand, for shells in the range covered by Table 1, the magnitude of the moment increases almost parabolically from the support to midspan as shown in Fig. 5, especially for the moment at the crown. At the valley, the moment increases at a slightly faster rate for smaller r/L values as can be seen by a comparison of the curves of Fig. 5 and 6. In determining the amount of transverse reinforcement for shells with r/L about 1.0, account should be taken of the greater curvature of the longitudinal distribution of M_ϕ.

CONTINUITY

The design constants of Table 1 are for simply supported shells, i.e., the supports are assumed to offer no longitudinal restraint. Thus, it will be found that taking the summation of the moment of T_x forces at midspan about any axis will equal $wL^2/8$. Nevertheless, the constants can be applied without any great loss of accuracy to shells continuous in the longitudinal direction. The effect of continuity, as one might expect from beam behavior, is to radically change the magnitude and sense of the T_x forces without affecting greatly the other internal forces such as T_ϕ and M_ϕ. However, while continuity alters greatly the longitudinal distribution of T_x forces, previous investigations have shown that only minor changes in transverse distribution occurs.

Without becoming involved in complex mathematics, a qualitative appraisal of the effect of continuity on the transverse distribution can be made by recalling that transverse distribution of T_x is solely a function of the relative proportions of transverse to longitudinal displacement. When, as in the case of a long shell, the vertical deflection of the edge, measured with respect to the crown of a unit strip at mid-

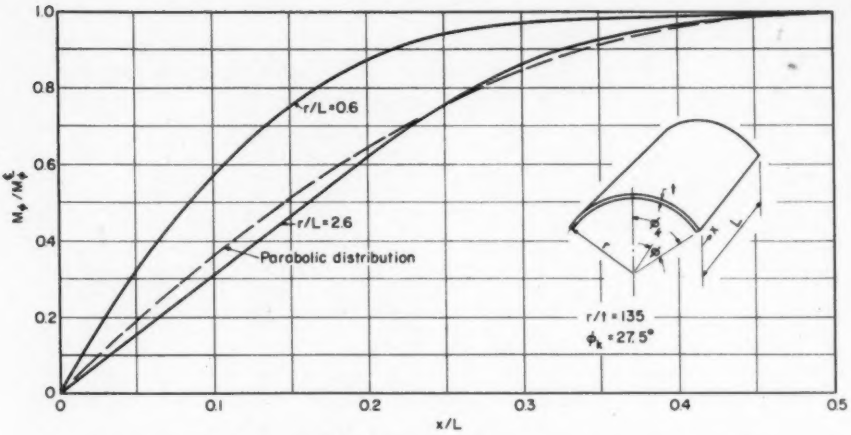


Fig. 6—Longitudinal distribution of transverse moment at valley

span, is small compared to the deflection of the same point measured longitudinally, the distribution of T_x in the transverse direction is linear and thus is similar to that of the fiber stress in a beam. As the relative displacement in the transverse direction to that in the longitudinal direction increases, the transverse distribution of T_x departs from a linear pattern, and becomes curvilinear with a decrease in the slope of the stress curve below the neutral axis. Since continuity decreases the deflection of the section at midspan with respect to the support, the effect of continuity is to increase the ratio of transverse to longitudinal deflection.

From this it follows that the transverse distribution of T_x forces in a continuous shell has slightly greater curvature than that of a simply supported shell of the same span and radius. An inkling of the relative difference between the two distributions can be obtained by comparing the design constants in Table 1 for any two r/L values with one r/L being 1.4 times the other. A plot of the two transverse distribution curves will show that while there might be significant change in the magnitude of T_x at the edge of the shell, the total area below the neutral axis will be about the same for both curves. In general, the difference will not be greater than 3 or 4 percent. Because of this, it is sufficiently accurate to use the transverse distribution of stresses of a simply supported shell, irrespective of the degree of continuity. As shown by Olsen,[4] the transverse distribution of T_x, for all practical purposes, is uniform throughout the length of the shell.

By similar deductive reasoning, the longitudinal distribution can also be accurately estimated. Because the transverse distribution is almost linear in long barrel shells, it is apparent that the magnitude of T_x at any section will be to the T_x in a simply supported shell, as the

ratio of the moment in a continuous beam of equal length and support condition is to the simple beam bending moment. For short barrel shells, because of the effect of shear strain, the longitudinal stresses over the support will be somewhat greater than that indicated by the analogy to a continuous beam. This increase, which will be slight for the range of shells covered in Table 1, is of little consequence since an underestimate of the intensity of the forces at the support will be compensated by an overestimate of the forces in the region of positive moment. Consequently proportioning the longitudinal forces on the basis of the variation of the moment occurring in a continuous beam can be applied without any decrease in the ultimate capacity.

The change in the transverse distribution of the T_x forces caused by continuity will naturally be reflected in the transverse distribution of the shearing forces. However, because a slight change in the location of the neutral axis occurs, the position of the peak shear will undoubtedly be quite insensitive to the effect of continuity, and, therefore, may be considered to occur at the same place as in a simply supported shell. On the other hand, the downward drift of the tensile forces will cause the shear curve to have more of a bulge near the valley. Since the shear stresses in this region are not generally the critical ones, inaccuracy in this area is relatively unimportant.

With respect to the longitudinal distribution of shear, the reasoning presented for T_x applies. Refinements aimed at increasing the accuracy of determining the intensity of the shear forces are hardly warranted in view of the common practice of providing shear resistance. Generally to avoid variable spacing, shear reinforcement is placed uniformly and thus leads always to overdesign because of the large number of bars crossing a section of principal stresses. For this reason, modification of the shear forces in a shell to correspond to the total shear in a continuous beam is satisfactory.

EXAMPLE

Notation

h = total vertical height of shell from edge to crown
y = vertical height of shell measured from edge
L = length of shell between supports
r = center line radius of shell
t = thickness of shell
w = load per foot of shell length
x = longitudinal distance measured from the left support
ϕ = angle measured from the right edge of shell
ϕ_k = angle subtended by the edge of shell measured from the center line axis
p_w = intensity of uniform load on unit area
p_d = intensity of dead load on unit area
T_ϕ = the direct force component in the transverse direction (considered positive when tensile)
T_ϕ^{CL} = T_ϕ at midspan of the shell
T_x = the direct force component in the longitudinal direction, (considered positive when tensile)
T_x^{CL} = T_x at midspan of the shell

S = the tangential shearing force (considered positive when it creates tension in the direction of increasing values of x and ϕ)

M_ϕ = the moment in the transverse direction (considered positive when it produces tension in the inner fibers)

$M_\phi^{CL} = M_\phi$ at midspan of the shell

The ease with which the internal forces can be computed makes the use of Table 1 self-explanatory. In all cases, the internal force is equal to the product of a multiplier and the design constants. The multiplier shown in the third row of Table 2 equals the product of the load times various powers of the dimension indicated in the heading of Table 1. However, to avoid misinterpretation, the computation required for a typical interior shell will be outlined. From the dimensions given in Table 2, $r/t = 45 \times 12/4 = 135$; $r/L = 45/50 = 0.90$.

Inspection of the constants in Table 1 show that there is only slight differences in the constants for values of r/t and r/L in the range with $\phi_k = 25$ and 27.5 deg. As such, the design constants will be selected from $r/L = 1.0$ and $r/t = 100$. But interpolation for the specific ϕ_k is recommended. To simplify this task, advantage will be taken of the fact that linear interpolation can be achieved by algebraically adding a fixed ratio of the two adjacent values. For this example, the constants for $\phi_k = 25$ deg are multiplied by $(27.5 - 26.4)/(27.5 - 25.0) = 0.45$, while the constants for 27.5 deg are multiplied by $(1.0 - 0.45) = 0.55$. Thus the design constant for T_x at the crown for $\phi_k = 26.4$ deg, $r/L = 1.0$, and $r/t = 100$ is $-(4.482 \times 0.45 + 3.509 \times 0.55) = -3.947$ which is recorded in the first row of numbers, second column of Table 2. The other coefficients are obtained in a similar manner.

In accordance with Eq. (1) and the intensity of load listed in Table 2, the equivalent dead load for which the shell is to be designed is

$$p_d = 50 + 30\left(\frac{0.44}{0.46}\right) = 79 \text{ psf}$$

The multiplier for T_x therefore is

$$\frac{L^2}{r}\, p_d = \frac{50^2}{45} \times 79 = 4390 \text{ lb per ft}$$

In a similar manner the other multipliers can be obtained as readily. The product of these and the tabulated constants gives the internal forces in the shell which appear in the columns marked Force.

A graphical representation of the tabulated values for T_x and M_ϕ is given in Fig. 7 for comparison with values as obtained by the beam method. As to be expected, the value of T_x as computed by the shell theory is slightly larger, while the value of the moment M_ϕ is slightly less. For design purposes the difference is negligible. However, this good agreement holds only for the interior shells. If the outer edge of

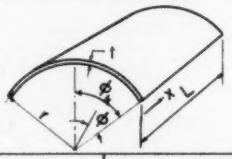
TABLE 2—CALCULATION OF FORCES IN A SIMPLY SUPPORTED INTERIOR CYLINDRICAL SHELL

Given:

$t = 4$ in. $\quad \phi_k = 26.4°$

$r = 45$ ft. $\quad p_u = 30$ psf

$L = 50$ ft. $\quad p_d = 50$ psf



Force	T_x		T_ϕ		S		M_ϕ	
Multiplier	$(L^2/r)p_d = 4390$		$(r)p_d = 3560$		$-(L)p_d = -3950$		$(r^2)p_d = 160{,}000$	
ϕ	Constant	Force (lb./ft.)	Constant	Force (lb./ft.)	Constant	Force (lb./ft.)	Constant	Force (ft.-lb./ft.)
ϕ_k (crown)	-3.947	-17,300	-1.441	-5130	0	0	-.00385	- 620
$.75\phi_k$	-3.455	-15,200	-1.204	-4290	1.415	-5590	-.00146	- 230
$.50\phi_k$	-1.551	- 6,800	- .595	-2120	2.571	-10,160	.00321	510
$.25\phi_k$	2.693	11,800	.083	300	2.759	-10,900	.00324	520
0 (valley)	10.111	44,400	.375	1340	0	0	-.00970	- 1550

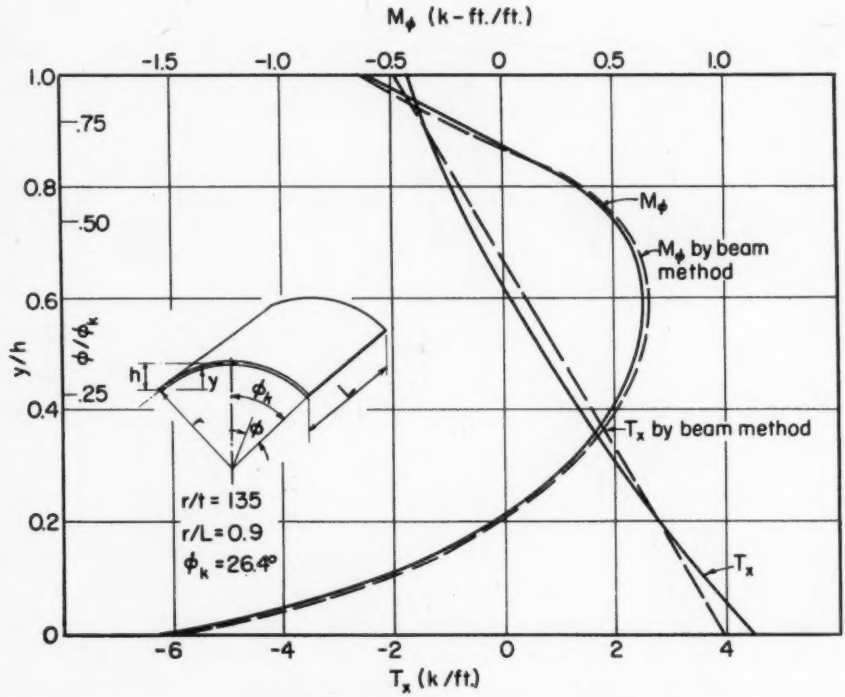


Fig. 7—Transverse distribution of T_x and M_ϕ for interior shell example

the exterior shell is not stiffened by an edge beam, marked increase in the intensity of T_x will occur at the edge.

If the shell is continuous in the longitudinal direction, the forces determined in Table 2 can be modified as previously discussed. For example if two 50 ft long shells are continuous over a central arch, the forces are multiplied by the ratio of the moments in a beam of similar continuity to the moment in a simply supported beam. Since the moment over a central support is $-wL^2/8$, obviously the ratio is -1.0. The ratio to be applied to the forces at midspan is

$$\frac{w\,L^2/16}{w\,L^2/8} = 0.50$$

Similarly, the shear forces are altered by the ratio of continuous beam shear at the interior support to that in a simple beam. The ratio is

$$\frac{5\,w\,L/8}{w\,L/2} = 1.25$$

The shear forces at the outer support are given by the following ratio:

$$\frac{3\,w\,L/8}{w\,L/2} = 0.75$$

As discussed above, continuity does not cause T_ϕ and M_ϕ to change significantly.

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American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1961, for publication in Part 2, March 1962 JOURNAL.

Las constantes de diseño para los cascos cilíndricos interiores de concerto

Par aumentar la confianza en el empleo del método de viga para el análisis de cascos cilíndricos interiores y también para proporcionar datos para los casos en que no es aplicable el método de viga, se presentan las constantes de diseño para los cascos cilíndricos interiores a base de la teoría de cascos. Estas constantes permiten la evaluación fácil de las fuerzas interiores en los cascos por medio de unas multiplicaciones sencillas. Además, se incluyen los estudios del efecto de la geometría del casco en la distribuición longitudinal de los esfuerzos. Una discusión del efecto de la continuidad aumenta la aplicabilidad de los constantes de diseño.

Constantes de conception pour coquilles cylindriques intérieures

Pour augmenter la confiance en l'utilisation de la méthode de poutres pour l'analyse de coquilles cylindriques intérieures, et aussi pour fournir des données pour ces cas oú la méthode de poutres ne convient pas, on présente des constantes de conception pour coquilles cylindriques intérieures, basées sur la théorie de coquilles. Ces constantes permettent l'évaluation facile des efforts internes dans la coquille par moyen de quelques multiplications simples. En outre, des études sur l'effet de la géométrie de la coquille agissant sur la distribution longitudinale des efforts, sont inclues. Une discussion de l'effet de la continuité élargit le champ d'application des constantes de conception.

Baukonstanten fuer innere zylindrische Betonschalen

Um das Vertrauen in der Anwendung der Traegermethode fuer die Analyse von inneren zylindrischen Schalen zu erhoehen, und um Daten fuer solche Faelle zur Verfuegung zu stellen, in denen die Traegermethode nicht anwendbar ist, werden hier Baukonstanten fuer innere zylindrische Schalen auf Grund der Schalentheorie dargestellt. Diese Konstanten ermoeglichen eine rasche Abschaetzung der inneren Kraefte in Schalen durch einige einfache Multiplikationen. Ausserdem wurden auch Untersuchungen ueber die Wirkung der Geometrie der Schale auf die Laengsverteilung der Spannungen aufgenommen. Eine Besprechung der Kontinuitaetswirkung erweitert die Anwendbarkeit der Baukonstanten.

CONCRETE BRIEFS

Notes from Field and Office

Investigation of Failure of Three Prestressed Concrete Piles

By CHARLES C. ZOLLMAN*

IT IS WELL-KNOWN that in the early stages of a construction job a contractor is beset by a multitude of minor difficulties, which usually straighten themselves out as the work progresses.

This was the case at the beginning of construction of the Patapsco Avenue Bridge and Causeway, Baltimore, Md. The project called for the use of a large number of prestressed concrete piles, and in the early stages of construction, three piles broke while being driven, for an apparently unknown reason. The owners became concerned, and called on our firm as prestressed concrete consultants to determine, if possible, the cause of the failure of the three piles. The following describes the investigation and its results.

SUBSTRUCTURE AND PILE DELIVERY

The substructure of the Patapsco Avenue Bridge and Causeway consists essentially of a number of concrete pile caps and prestressed concrete piles. The prestressed concrete piles for the project are shown in Fig. 1, and were manufactured in Wilmington, N.C. The piles were transported by barge to Baltimore, and were temporarily stored in the contractor's yard at the unloading dock, which is located in the vicinity of the project. The piles were then loaded on pole trailers and transported to the erection site, where they were again stored until called for in the erection schedule.

NATURE OF THE PROBLEM

According to the information furnished to the consultants, three of the first forty-four 16 in. sq prestressed concrete piles broke during driving. Two of these three piles were 54 ft long and failure occurred 30 ft 6 in. from the top in one case and 31 ft 3 in. from the top in the second. The third pile was 64 ft long and breakage occurred 49 ft from the top. The soil in which these piles broke is shown on the boring profile in Fig. 2. The term breakage as used here is understood to be a complete break of the pile, necessitating its removal. Breakage of the three piles occurred below grade when the driving was almost completed and was suspected to have occurred when there was a sudden decrease in the number of blows per foot. After extraction of the piles from the ground the suspected total break was confirmed.

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*Member American Concrete Institute, Partner, Schupack and Zollman, Consulting Engineers, Newtown Square, Pa.

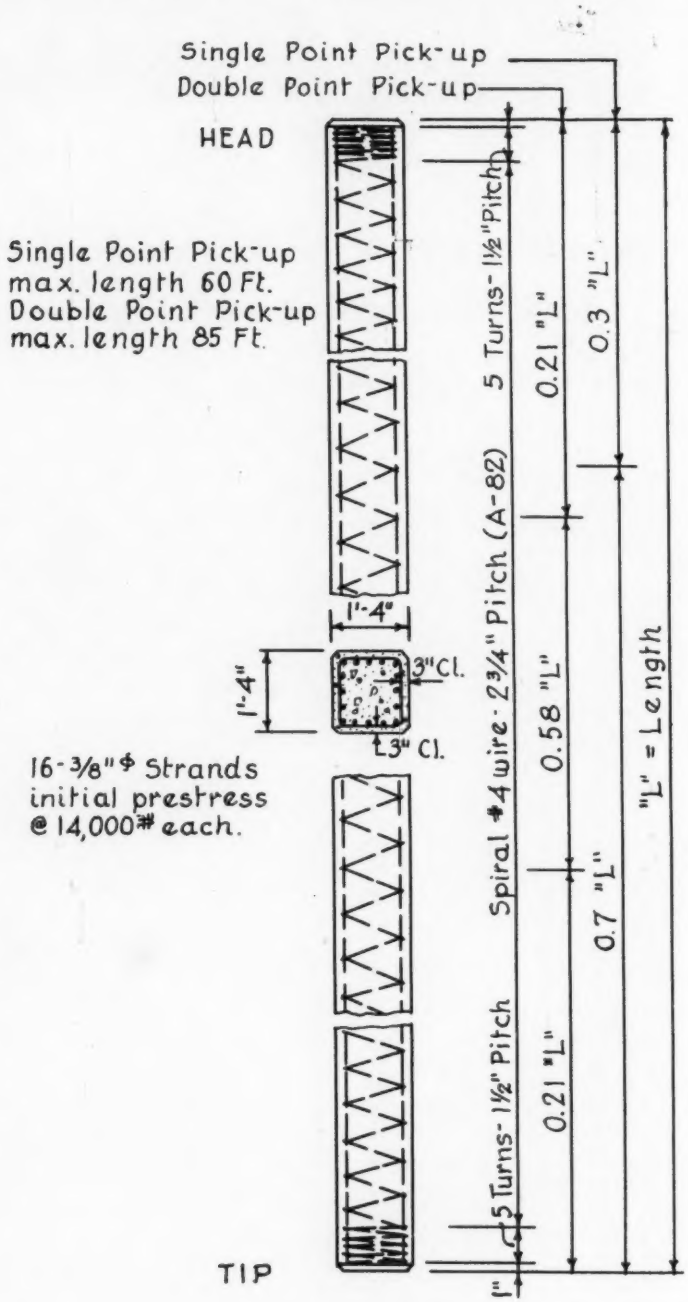

Fig. 1—Pile detail

PAST EXPERIENCE WITH DAMAGE OF PILES

Damage to prestressed concrete piles during driving has been found to fall primarily into two categories:

1. Excessive spalling at the top of a pile

2. Horizontal cracking occurring completely around the pile at a plane about 3/10 of the distance from top.

Spalling at the top of a prestressed concrete pile usually occurs when the end of the pile is not perfectly square with the axis, if the cushioning is improper, if the driving head is not properly cleaned of all burned cushioning material or if strands protrude too far beyond the end surface of the pile. Preventive measures can be taken by insuring that the end of the pile is square or by using, for example, oak cushions under the ram and pine cushions at the head of the pile: by properly chamfering the edges of the pile; by recessing the ends of the prestressing strands within the pile head; and by properly cleaning the driving head of all old burned out cushioning material. These preventive measures have in the past substantially reduced spalling at the heads of piles during driving.

The cause of horizontal cracking at a plane about 3/10 of the distance from top has been traced to driving heads held rigidly in leads by snugly fitting guides (in other words, by tight-fitting heads) combined with rapid blows, usually from a double-acting hammer. The snug fit of the cast steel driving head around the head of the pile, and the inability of the pile to fully recover from succeeding blows, induces torsional stresses (see Fig. 3). These stresses in turn result in the concrete cracking, as shown in Fig. 3. In all such cases inspection of the head of the pile reveals scars where the driving head has jammed against the faces of the pile.

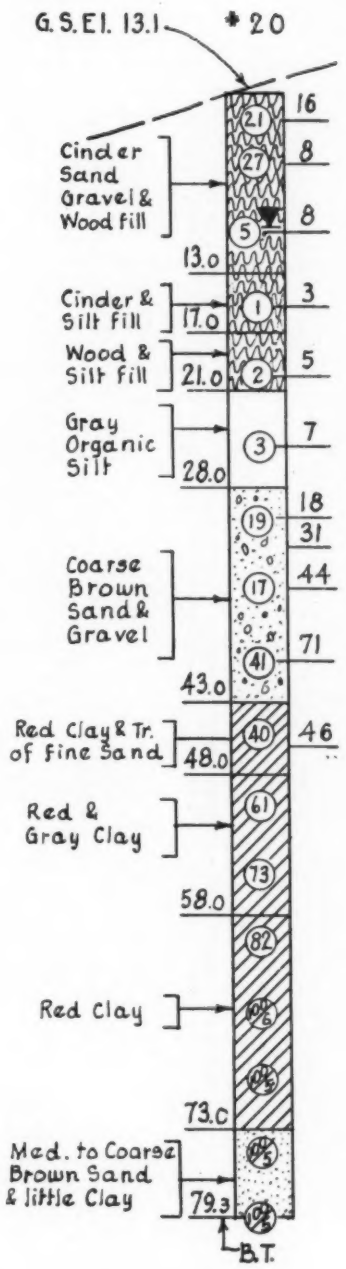

Fig. 2—Boring profile

Inasmuch as these two difficulties are those most commonly encountered in the use of prestressed concrete piles, the first efforts of the consultants were directed toward determining whether any of the above physical indications were to be found.

OPERATIONAL CONDITIONS AT JOB SITE

Driving head of hammer

On inspection it was found that the driving head of the hammer used was loose fitting and allowed for slight rotation of the head of the pile during driving. The effect of this was noticeable during the driving since rotation of the driving head of the hammer occurred. Because of this rotation, no torsional stresses could have been induced in the piles. The allowance for rotation in the driving head was also supported by the fact that on close inspection it was found that none of the heads of the driven piles, whether broken or not, showed any trace of binding between the driving head of the hammer and the heads of the piles.

Cushioning material

Plywood was used as cushioning material during the early stages of the driving process, but was subsequently replaced by three 1-in. layers of "gum," which is a soft wood. It is believed that this change was an improvement because past experience in driving of prestressed concrete piles has indicated that plywood does not sufficiently absorb the blows of the type of hammer used at this project. While the three layers of gum might be sufficient, it was believed that a four-layer cushion of gum material would further improve the cushioning. For the longer piles it was recommended that a five-layer cushion be used, which would be about 6 in. thick before any compression of the cushioning material has taken place. A 6-in. pine cushion at the heads of the piles has proved most successful in several jobs in reducing spalling of the concrete at the heads

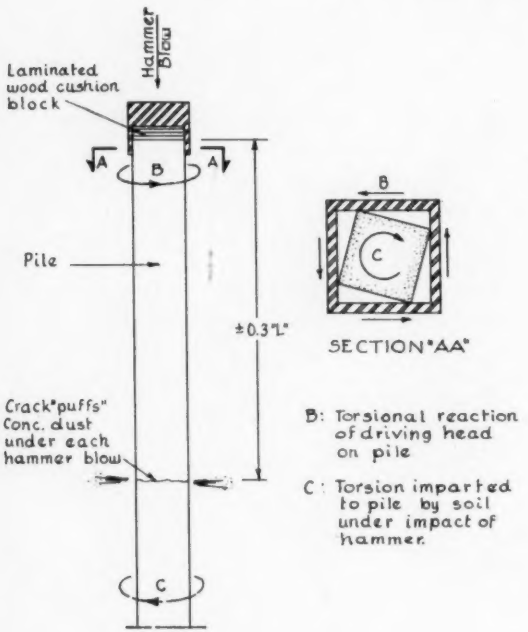

Fig. 3 — Diagram showing how a driving head imparts torsion to piles

of piles. It should be noted that no spalling of concrete occurred at the heads of the piles driven during the visits to the job site. At that time three layers of gum material were used for a number of piles, and for the last five piles (70 ft long) four layers of gum were used.

Examination of broken piles

Insofar as could be determined neither the head nor the tip of the broken piles bore markings of any type which would indicate that damage or binding had taken place. The type of break appeared to resemble more closely a concrete cylinder break which has occurred under compression, rather than a break caused by tension.

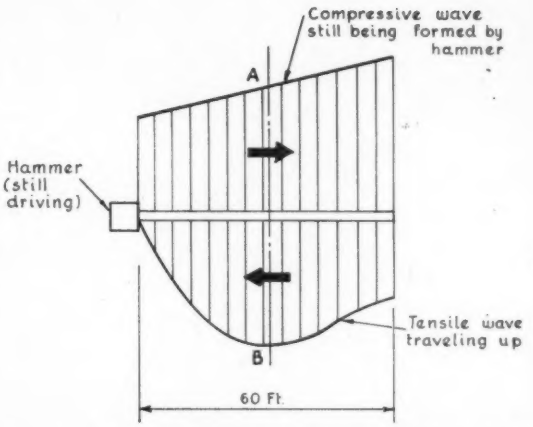

Fig. 4 — Wave formation and reflection in 60-ft pile in soft ground. Note: Reflected tension wave B is always overlapped by a compressive wave as at A

Examination of testing reports

(a) Concrete—The reports submitted by the testing laboratory were reviewed. The testing results, as indicated by the laboratory reports, were consistent over an extended period and met the contract specifications.

(b) Strands—The reports from the mill furnishing the strands used in this project indicated that the strands met the contract specifications for this type of material.

CONCLUSIONS

1. Since the driving head on the hammer used was loose-fitting and allowed slight rotation of the head of the pile during driving; since proper cushioning appeared to have been provided; since no scars caused by binding were found on any of the piles; since the breaks occurred at the midpoint or in the lower half of the piles, rather than in the upper half; and since the type of breakage was substantially different from that caused by torsion, it was concluded that the failure of these particular piles was not due to torsional tensile stresses.

2. The fact that only three out of the first 44 piles were broken, no breakage occurred during the driving of subsequent piles, and the testing reports were satisfactory, would indicate that the breakage was not due to an inherent shortcoming in either the piles themselves or the driving method used. The percentage of broken piles amounted to less than 3 percent, which is a normal allowance for breakage in the driving of any type of pile.

It was concluded then, that the breakage of these piles was accidental in nature.

A double-acting diesel pile hammer was used at this job site. The manufacturer's catalog indicated that the maximum energy per blow, for a stroke of 8 ft, was 22,400 ft-lb, and that the average applied energy per blow was 16,800 ft-lb. However, it has been reported by contractors who have used this particular pile hammer for other projects that under hard driving conditions the applied energy per blow can be as high as 30,000 ft-lb or more. Should this have happened during this project the delivered energy might have been almost twice the weight of the piles which were broken, since these

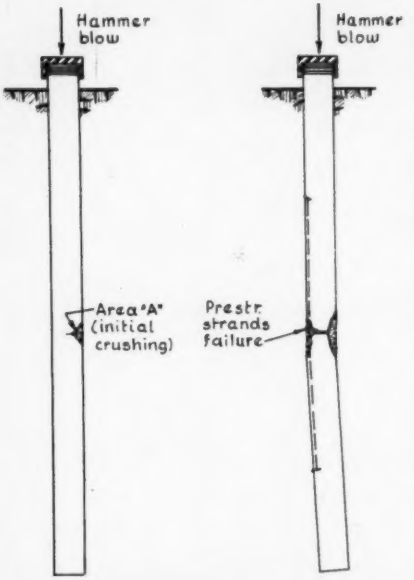

Fig. 5—(a) (left) Crushing failure (b) (right) tensile failure

piles weighed approximately 14,000 and 17,000 lb. With this ratio of energy to weight, difficulties have been experienced in the past.* In a short pile of, say, 50 to 70 ft, the peak of the reflected tension wave is always overlapped by a compressive wave because the wave length is greater than twice the length of the pile (see Fig. 4). When the point of a pile reaches hard material the wave caused by the hammer blow is reflected from the point of the pile as a compressive wave and not as a tensile wave. Too severe a hammer blow may cause compressive failure. At this job site, increased resistance was encountered during jetting in the area of the broken piles after a certain depth was reached. This increased resistance may have been caused by the layer of coarse brown sand and gravel which is indicated at Boring No. 20 from Elevations 28 to 43 (see Fig. 2). To penetrate this layer, harder driving was required. A possible local deficiency of the concrete, due either to the manufacture of the pile or mishandling or improper transportation, combined with hard driving of the double-acting hammer, may have initiated crushing in Area A as shown in Fig. 5a. The situation shown in Fig. 5b could then have resulted, since continuation of the heavy dynamic pounding may have caused tensile failure of the strands. There are several other possibilities, of course, one of which could have been that nonuniform jettying may have loosened the soil irregularly, thus causing eccentric driving of the pile. In the piles in which the strands were squashed, a plane of weakness may for similar reasons have caused crushing of the concrete, with subsequent squashing of the strands.

A final comment perhaps could be made that, as stated before, minor difficulties are bound to arise at the beginning of any project. For example, a number of details in the driving operation may initially appear unimportant, but their bearing on the success of the complete operation is in reality of utmost importance, and this becomes quickly apparent as the work progresses. In this particular case, such difficulties resulted in the breakage of three piles. As a consequence, more care throughout the casting, handling, transportation, and driving operations was no doubt exercised, and this may have been a contributing factor to the cessation of the pile failures.

In view of the findings reached during this investigation, the consultants concluded that future breakage would not exceed the maximum allowable percentage of ± 3 percent.

The Patapsco River Bridge and Causeway is being built by the McLean Construction Co., for the City of Baltimore. Bernard L. Werner is the director of public works, George V. Walters is highway engineer and Louis B. Kravetz is the city bridge engineer.

*These difficulties are explained in "Tension in Concrete Piles During Driving," E. A. L. Smith.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Road bridges of Serre-Poncon (in French)

M. COURBON, *Travaux* (Paris), No. 308, June 1960, pp. 449-459

Reviewed by ETIENNE D. ROLIN

Describes four long span concrete bridges built by the cantilever method to minimize shoring. The roadways are all of prestressed box sections. The first and second bridges consist of successive double cantilevers with special connections at the extremities to develop continuity under live loading. The third bridge is a single 160-ft cantilever during construction only. Its extremity is then jacked up on the abutment to impose a reaction on it. The fourth bridge consists of a cantilever from each abutment with the extremities hinged. The entire abutments are then set free to rock through the removal of some temporary supports to accomplish a three-hinged arch.

Precast prestressed concrete bridges. 2—Horizontal shear connections

NORMAN W. HANSON, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 2, May 1960, pp. 38-58

AUTHOR'S SUMMARY

Presents an initial study of composite action between precast girders and a situ-cast deck slab. Sixty-two push-off specimens and ten T-shaped girders were tested to explore various means of horizontal shear transfer at the contact surface between precast and situ-cast concrete in composite construction. Adhesive bond, roughness, keys, and stirrups were included as test variables. Experimental results are given in terms of a basic shearing strength related to the concrete-to-concrete joint characteristics, plus a term related to the percentage of stirrup reinforcement.

Precast prestressed concrete bridges. 3—Further tests of continuous girders

ALAN H. MATTOCK and PAUL H. KAAR, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 3, Sept. 1960, pp. 51-78

AUTHORS' SUMMARY

This part reports an experimental program designed to provide further information regarding the feasibility of establishing continuity between precast girders in adjacent spans. It was concluded that it is possible to produce continuity, and adequate negative support moment resistance for both static and dynamic loads, by placing deformed bar reinforcement longitudinally in the deck slab over the interior supports. Also reported is a brief investigation of two simple structural details designed to provide continuity between precast girders when reversal of moment occurs at supported sections. It was concluded that the details described can provide an adequate resistance to positive moments of the magnitude likely to occur at support sections of multispan con-

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tinuous precast-prestressed concrete bridges.

Nine span bridge but no expansion joints

H. W. Stephenson, *Engineering* (London), V. 190, No. 4924, Sept. 2, 1960, pp. 308-309
Reviewed by Aron L. Mirsky

Qurnah bridge, on the Baghdad-Basra all-weather highway over an old bed of the Euphrates, is an example of structural design to meet existing conditions. The nine spans (two at 90 ft, seven at 120 ft) are fully continuous, of steel with all connections made with friction grip bolts; deck is semicomposite, acting as sway bracing but taking no girder stresses—this was accomplished by breaking bond of outer thirds of 60-ft panels. Piers (reinforced concrete on precast bases) and abutments (box type, filled with local brick laid in courses) are on reinforced concrete piles. Good concrete was made with local sand containing up to 3 percent sulfates, by washing it, restricting it to 40 percent of the coarse aggregate, and using a water-cement ratio of 0.36 to 0.46.

Structural behaviour of highway bridge decks

S. D. Lash and B. B. Hope, *Engineering Journal* (Montreal), V. 43, No. 11, Nov. 1960, pp. 88-99
Reviewed by Aron L. Mirsky

Decks of three bridges were tested under static loads: two, tested in the field, consisted of 6 in. reinforced concrete slabs on steel stringers spaced 3 ft 3 in. on centers; the third was a laminated timber deck on steel stringers tested in the laboratory where the bridge was re-erected.

Results indicated that the concrete slabs actually acted as composite decks to a considerable degree, even though no shear connectors were installed. Maximum stresses and deflections in the stringers were thereby reduced. Experimental values of deflections and bending moments and theoretical values, obtained by the harmonic analysis method of Hendry and Jaeger,* were in good agreement, validating the use of this method for estimating lateral distribution of wheel loads. Authors conclude current specifications are unduly pessimistic regarding load capacity of such bridges.

*Hendry, A. W., and Jaeger, L. G., *Analysis of Grid Frameworks and Related Structures*, Chettoe and Windus Ltd., London, 1958; Prentice-Hall, Englewood Cliffs, N.J., 1959.

Construction

Regulating works of the Baix-le Logis Neuf canal by La Compagnie Nationale du Rhone (Les travaux d'aménagement de la chute de Baix-le Logis Neuf par la Compagnie Nationale du Rhone)

G. Gres, *Le Génie Civil* (Paris), V. 137, No. 4, Feb. 15, 1960, pp. 77-84
Reviewed by Aron L. Mirsky

This paper, a continuation of previous article [*Le Genie Civil*, V. 136, No. 9, May 1, 1959, pp. 193-200; "Current Reviews," ACI Journal, V. 32, No. 1, July 1960 (*Proceedings*, V. 57), p. 109)] discusses the barrage at Loriol and the entrance to the canal.

Modern quays (in Dutch)

J. P. Josephus Jitta, *Cement* (Amsterdam), V. 12, No. 9, June 1960, pp. 795-796
Reviewed by John W. T. Van Erp

Concrete quay construction consisting of a platform supported only by vertical piles is described. Anchorage at the land side end is by a vertical concrete apron embedded in the soil or in the stone revetment. The use of precast elements is preferred especially where deterioration by frequent salt water contact is a possibility. Fenders are used to protect against heavy impact of ships. Elastic fender-platform connection is described.

30 story concrete building in Brussels (in Dutch)

G. Colz, *Cement* (Amsterdam), V. 12, No. 11, Oct. 1960, pp. 976-978
Reviewed by John W. T. Van Erp

Building is entirely cast-in-place concrete, and being only 50 ft wide needed considerable windbracing. This is taken care of by concrete partitions

and elevator shafts, vertically continuous. Exterior walls were mostly glass with practically no columns. Floors are of prestressed steel beams embedded in concrete and then released. Result is prestressed structural steel and concrete floor of shallow height. Two expansion joints separate building into three units 65 ft wide. Total quantity of concrete used was 39,000 cu yd, making it one of the biggest buildings of this height.

On the construction of the research reactor near Munich (Vom Bau des Forschungsreaktors bei Muenchen)

NIKOLA DIMITROV, *VDI Zeitschrift* (Düsseldorf), V. 102, No. 24, Aug. 21, 1960, pp. 1140-1148

Reviewed by ARON L. MIRSKY

Based on the swimming-pool reactor at Oak Ridge, this German research reactor, modified to suit local conditions, has an outer shell in the form of a half ellipsoid of revolution (semidiameter of base 15 m, height 30 m) in reinforced concrete. Paper discusses, *inter alia*, the stress analysis (membrane theory was found applicable since the boundary perturbations were quickly damped out), crane load, internal overpressure, and wind stresses, and radiation protection.

Estimator's general construction manhour manual

JOHN S. PAGE, Gulf Publishing Co., Houston, Tex., 1960, 218 pp., $10

Presents a convenient method of estimating direct labor for complete general construction work in any given system, plant, or location. The manhour tables in the manual contain numerous listings.

The manual points out how to arrive at composite rate using productivity efficiency and production elements. With the composite rate, man-hour estimating can be applied to any general construction job.

Subject areas covered in 17 sections include: concrete roadways, sheet and foundation piling, formwork, reinforcing steel and mesh, miscellaneous embedded items, concrete, masonry, special flooring, special walls and ceiling, roofing and siding, scaffolding, and others.

Construction Techniques

Formwork to concrete

C. K. AUSTIN, Cleaver-Hume Press Ltd., London, 1960, 283 pp.; available in the United States from Simmons-Boardman Publishing Corp., Book Division, New York, $5

Reviewed by MARY K. HURD

The author is lecturer in carpentry, joinery, and formwork at the College of Technology, Bristol, and it appears that his book is directed primarily to builders of forms, but the wide scope of its layout and detail drawings offer much material of interest to a broader field. Opening on a practical note of explanation that "good shape at the concrete face" is more appropriate to form construction than external neatness, the 26-page introduction is crowded with condensed information on form planning to save time and materials, requirements of good formwork, materials and their allowable stresses, and methods of calculating loads and pressures on the forms.

Simplified design formulas, techniques for measuring and leveling, the stripping sequence, and the carpenter's tool kit are also treated in the introduction.

In the main body of the text, line drawings, almost always facing the pertinent text material, present construction details of forms for foundations, columns, beam and girder floors, curved beams, suspended floors and walls, and layout for multistory forming. Brief design calculations are shown for column yoke spacing, slab sheeting and its joist support, beam bottom forms, shores, and wall forms. Lateral pressure used for design of column yoke spacing is hydrostatic head based on a weight of 125 lb per cu ft of concrete, with no maximum limiting value stated. Lateral pressures used in wall form design are those recommended by M. L. Elkins (the same as stated in design assumptions for the Keeley Plyform Calculator, derived from a

1946 letter by Universal Form Clamp Co.). For vibrated concrete these pressures are given for placement rates of 2, 3, 4, 5, and 6 ft per hr, at temperatures of 50 and 70 F. No maximum limit is stated, nor is it clear what pressure is advised if placing conditions are beyond these rates or temperatures.

Coverage of other topics—including moving forms; tank, chimney, and tower forms; forms for arches, groined vaults, cove ceilings, balconies, and canopies; and forms for culverts, conduits, and roofs with dormers—is confined to detail drawings and layout and construction suggestions, with attention to practical geometry for the carpenter's use.

Bulk of the text is based on timber construction of formwork, but concluding sections on patented steel forms and steel scaffolding and shoring indicate some of their potential applications. Well-handled illustrations and context will carry the American reader over such British terminology as "upstand," "respond," and "making good," and the author's definition of "shuttering" makes that term seem eminently logical.

Shaping a two-acre sculpture

Architectural Forum, V. 113, No. 2, Aug. 1960, pp. 118-123

Reviewed by MARY K. HURD

A skillful job of form building dominated the construction scene as concrete was placed for Saarinen's new TWA terminal at Idlewild Airport. Free-form roof of lightweight concrete ranging from 7 to 40 in. in thickness is supported on four giant buttresses of heavily reinforced conventional concrete.

Contractor made elaborate computations and hundreds of detail drawings in planning forms which were allowed to deviate no more than ¼ in. from the architect's plans. In forming the 2-acre roof, tubular steel shores and scaffolding were first erected on a carefully designed grid system; then curved ribs and beams were fitted with the help of prefabricated wedges into the U-shaped clamp at the top of each vertical member. Wood sheathing was nailed across ribs, kerfed and soaked as needed to fit curvature. Buttresses were formed with pre-built panels assembled in place on the job.

Excellent photographs showing form details are supplemented by excerpts from the architect's and contractor's plans.

Spray concrete simplifies rib shell forming

Building Construction Illustrated, V. 30, No. 4, Oct. 1960, pp. 30-31

Shotcrete provided an economical means of concreting difficult-to-form cantilevered portions of a zoo structure in Brookfield, Ill. Cost was reported to be $1.00 per sq ft for an average 3-in. shell thickness.

Adjustable pipe braces speed erection of columns

Contractors and Engineers, V. 57, No. 11, Nov. 1960, p. 9

Inexpensive pipe brace, already tested in Holland, Mich., used for holding forms and speeding up erection of precast and prestressed members.

Old bridge provides formwork for new

Engineering (London), V. 190, No. 4931, Oct. 21, 1960, p. 546

Queen's Bridge Perth

The Engineer (London), V. 210, No. 5465, Oct. 21, 1960, p. 683

Reviewed by ARON L. MIRSKY

Bridge over the Tay, Scotland, is composed of four longitudinal beams, with transverse slabs at top and bottom, forming three hollow boxes side by side, prestressed by the Freyssinet system and continuous over three spans (85.6, 187.4, and 113.1 ft). Since river is tidal and subject to large and rapid fluctuations in level, scaffolding was ruled out; formwork for new structure was suspended from jacked-up skeleton of old four-span steel girder bridge.

Sealing concrete and reinforced concrete constructions in sanitary engineering (in Polish)

W. Danilecki and J. Pluta, *Inzynieria i Budownictwo* (Warsaw), No. 10, 1959, pp. 436-443

Polish Technical Abstracts No. 2, (38), 1960

Discusses: conditions for erecting watertight buildings (silos, settling tanks, etc.); studies on the suitability of PVC slabs for sealing expansion joints in concrete constructions; the computation of dimensions of expansion bands; practical application of PVC bands in structures erected in Poland; the effect of the static scheme of construction on the appearance of cracks in concrete and on watertightness.

New forming method for retaining walls

Bill Allen, *Contractors and Engineers*, V. 58, No. 2, Feb. 1961, pp. 12-13

Reviewed by Mary K. Hurd

Forms are assembled entirely on the job site, using unframed 4 x 8 ft plywood panels and vertical wales made of 2 x 4 ft stock. There are no studs, and nailing is held to a minimum. Snap ties are set 2-ft on center both ways through predrilled holes in the plywood. Horizontal 1 x 4 ft members support the tie wedges.

Forms are braced by struts to the ground on only one side of the wall. Illustrations show wall forms 16 ft or higher; they are said to be good for concreting rates up to 8 ft per hr.

Notes about exposed exterior concrete (in Dutch)

H. J. Engel, *Cement* (Amsterdam), V. 12, No. 9, June 1960, pp. 777-778

Reviewed by John W. T. Van Erp

As the use of exposed concrete for high quality work increases, it is more than ever necessary to adhere to the best practices in design and execution. This concrete has to look good now and in the future, and its exposure to sun, rain, and frost is bound to put high requirements on its quality. A dense, perfect surface, a minimum steel coverage of 3½ cm (1⅜ in.) crack width not over 0.2 mm (0.008 in.) are among the requirements. Other points: rigid reinforcing placed in the right location; proper laps of bars; enough placing room at crowded reinforcement (column ends at beams), rigidity of formwork, especially where vibration is used; and cold joints and expansion joints. Author warns against careless execution which may impede the good reputation of concrete.

Use and abuse of scaffolds

Richard N. Vandekieft, *Transactions*, 1960 National Safety Congress, V. 8, Construction Industry-Public Employee, pp. 13-16

Reviewed by Mary K. Hurd

Steel frame scaffolding was originally developed some 25 years ago to support relatively light loads, primarily those involved in getting workmen and some materials to the work area. Primary concern was not for strength of the structure, but for its stability and the soundness of the foundation on which it rested. Safety rules largely dealt with counteracting external forces.

Adaptation of this scaffolding in recent years to support concrete formwork requires correct design for the relatively heavier loads now being carried by the tubular members. Careful design of the formwork being supported is also important, because excessive deflection of a form member bearing directly on a scaffold leg introduces bending in the tubular column, seriously reducing its load capacity.

Safety requirements include: solid, even footing; accurate, plumb assembly of scaffold; good condition of scaffolding members; careful centering of loads on individual members; and continuing inspection during concreting. Allowance for slab deflection during reshoring is recommended.

Author alludes to plans of the recently organized Steel Scaffolding and Shoring Institute for developing design criteria, safety rules, and standard testing procedures for scaffold and shoring equipment.

Blast furnace cement silos for ENCI (in Dutch)

P. M. KNOLS, *Cement* (Amsterdam), V. 12, No. 9, June 1960, pp. 757-760
Reviewed by JOHN W. T. VAN ERP

Group of eight silos, 22 ft high, cast in conventional forms. Walls, 7 in. thick, were prestressed vertically and horizontally. Horizontal cables span a complete circumference (360 deg) but stressed ends alternate 180 deg. Friction coefficient for losses was figured at 0.25, but measured at 0.27. Vertical prestressing mainly to take care of temperature stresses and stresses during partial filling; silo bottoms of steel plate. Design calculation was based on cement content considered as a fluid of specific gravity 0.9.

Failures of slender steel tubular frames

C. W. BORDEN, JR., *Transactions*, 1960 National Safety Congress, V. 8, Construction Industry-Public Employee, pp. 6-12
Reviewed by MARY K. HURD

Stresses the importance of bracing, strutting, and stiffening of sections to maintain rigidity of lightweight tubular steel structures such as those now commonly used to support formwork for concrete. Design of these structures must anticipate lateral forces as well as buckling caused by axial overloading. Considerations for design of individual members of such a framework are given.

Using the Euler formula, author shows how failure may occur in a tower made of tubular members when only one strut is omitted. Wherefrom he concludes: no strut or brace should be left out of such a structure in the field, or if temporary omission is required, a substitute of equal strength must be provided; compression members which have buckled or deflected should not be straightened in the field for reuse; and all bolts must be in place and tightened in field connections to assure continuity of structure.

Two cases of tubular steel scaffolding-shoring failures in concrete construction are analyzed. In one case, failure was caused by excessive column loads due to errors in estimation of the unsupported column length. Another case involved formwork for the inclined portion of an arch overpass structure. Pipe columns here were adequately designed for vertical loads, but no inclined bracing had been provided to counteract the lateral load component due to hydrostatic force of wet concrete against the inclined arch surface. Further, columns were simply wedged at the arch form surface, and were not nailed, tied or otherwise restrained from lateral movement.

Explaining why eccentric loading is more critical with tubular steel members than with timber posts or shores, the author makes important, practical suggestions for checking design, erection, and moving of tubular steel supporting members. Ideas in this paper are a safety "must" for anyone using or about to use patented steel scaffolding and/or shoring to support concrete formwork.

Design

Deformations

English translation of *Bulletin D'Information* No. 16, Comité Européen du Béton (Paris), Mar. 1959, 27 pp. (Limited supply of single copies available from ACI headquarters.)

A translation of a report of Comité Européen du Béton Committee No. 4 prepared by the Portland Cement Association for distribution by the ACI-CEB Collaboration Committee.

The CEB Committee summarizes the currently valid specifications on the computation of deflections and on allowable deflections in the various CEB member countries and makes a comparative evaluation of these specifications. A study was undertaken of the factors affecting deflection, as revealed by evaluation of test evidence and a study of a number of recommendations for improvements in the analysis of deflections as presented by various authors and committees.

On the basis of these extensive preliminary investigations, recommendations are made by the committee for

the computation of deflections in a manner more closely in agreement with reality as well as for the limitation of deflections in reinforced concrete structures. (The report does not deal with deflections under test load.)

Using finite difference equations to find the stresses in hypar shells

N. C. Das Gupta, *Civil Engineering and Public Works Review* (London), V. 56, No. 655, Feb. 1961, pp. 199-201

Author's Summary

The solution using finite difference approximations of two simultaneous differential equations obtained from equilibrium and compatibility conditions for a hyperbolic paraboloidal shell is the subject of this paper, which also contains a numerical example of a rectangular hypar shell with rigidly clamped edges.

Successive approximations by electronic computer (in Dutch)

D. Keijzer, *Cement* (Amsterdam), V. 12, No. 7, Feb. 1960, pp. 607-608

Reviewed by John W. T. Van Erp

Time consuming design calculations by reiterative methods can be accomplished by electronic computer. Description of operation in detail is given, step by step sequence of analysis, how data are to be programed into computer. Check methods at various stages of calculation are given, also time and cost of calculation and possible maximum scope of design problem.

Membrane stresses in hyperboloid shells of revolution

Placido Cicala, *Proceedings*, ASCE, V. 86, No. EM5, Oct. 1960, pp. 147-159

Author's Summary

The computation of membrane stresses and of the corresponding deformations for a hyperboloid shell of revolution under the most general loading is reduced to the evaluation of simple integrals. For the determination of wind effects, diagrams capable of simplifying computations are presented and an approximate formula is suggested.

Stresses and deflections in helical staircases

B. M. Annette and P. B. Morice, *Civil Engineering and Public Works Review* (London), V. 55, No. 652, Nov. 1960, pp. 1423-1427

Describes a computer program which gives the stress resultants and deflections at intervals along uniform helical girders, such as those often used for helical staircases, set up by uniform or concentrated loads. Some results are given to indicate the effect of different relative section thicknesses.

Matrix formulation of the folded plate equations

A. C. Scordelis, *Proceedings*, ASCE, V. 86, No. ST10, Oct. 1960, pp. 1-22

Author's Summary

An analytical procedure, utilizing matrix algebra, is developed for determining longitudinal stresses, transverse moments, and vertical deflections in folded plate structures. The procedure provides an efficient and systematic method for analyzing structures of this type. The sequence of matrix operations can be readily programed for a digital computer having available matrix subroutines. A sequence which has been programed for the IBM 704 digital computer is described.

Calculations of reinforced concrete sections (Calculo de secciones de hormigon armado)

Miguel Garcia Ortega, *Bulletin*, No. 187, Consejo Superior de Investigaciones Cientificas, Instituto Técnico de la Construcción y del Cemento, Madrid, 1959: Part 1, 83 pp.; Part 2, 119 pp.

Reviewed by Celso A. Carbonell

The calculation of reinforced concrete sections is based on certain basic facts obtained from general conditions of equilibrium between the variables involved in the problem. In each case some of these variables are constant, and the others must be worked out from the basic relations between the variables and the known facts.

But the basic relations are not the same for all cases; even assuming the same elasticity hypotheses, the equations differ for the cases of compression

and simple bending, or for the type of section. Further complications arise as a result of the modern tendency to calculate the stress in the concrete from laboratory experiments rather than from linear stress-strain relationships. To each hypothesis there corresponds a particular set of equations for simple bending and for compression, and also for each of the types of sections considered in practice.

From the foregoing it is evident how important it is to establish general relationships, applicable in all cases, needing only the introduction of the particular variables.

Part 1 seeks to establish these general relations or fundamental equations, and to consider their particular applications. In spite of the complexity of the problem and the large number of variables, the relationships which have been obtained are extremely simple. There are two factors which have particular values for each case, depending on whether it is bending or compression, and depending on the shape of the section and the assumed distribution of the stress in the concrete.

The properties and significance of these two factors are considered in detail, and also the manner of determining their values for any possible particular case. In the most important cases, a detailed study of the problems is made, and the values of these factors are worked out. Formulas, tables, and charts are provided. Methods are also given for obtaining easily the charts, tables, and formulas corresponding to other cases, when these are assumed to be of interest, due to the particular type problem involved.

As particular cases, bending and compression have been considered separately. Two elastic laws have been assumed: Hooke's law, and the parabolic stress distribution law of Brandtzaeg. Three types of cross section have been considered, rectangular, circular, and T-shaped, and two types of steel, ordinary and "Tetracero."

Part 2 contains the tables and charts to be used in conjunction with Part 1.

Materials

Recommended specifications for water-reducing retarders for portland cement concrete

Public Roads, V. 31, No. 6, Feb. 1961, pp. 153-154

Describes types of retarders, gives acceptance requirements for approval of admixtures and performance requirements, and lists applicable ASTM and AASHO methods and specifications.

Epoxy resins — Their applications and technology

HENRY LEE and KRIS NEVILLE, McGraw-Hill Book Co., New York, 1957, 308 pp., $8.00

Presents detailed information on chemical composition and physical properties of epoxy resins. Discusses the industrial applications of resin systems in laminates, adhesives, coatings, castings, and foams. Application techniques, while complete for many uses, are meager for concrete repair or joining of structural members.

Some physical aspects of the hydration of portland cement

T. C. POWERS, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 3, No. 1, Jan. 1961, pp. 47-56

AUTHOR'S SUMMARY

The hydration of portland cement requires a doubling of the volume of solids in a specimen of paste. Half of the reaction products are deposited inside the boundaries of the cement grains, and half outside, simultaneously. The process requires an insoluble material to remain in solution for great lengths of time. Also, the process occurs without much dilation of the specimen. These aspects of hydration present a problem of understanding not readily explainable in conventional terms of chemistry. An explanation is given, showing how the chemical processes are controlled by physical conditions peculiar to a system giving a gel as reaction product.

Who invented portland cement?

HAROLD H. STEINOUR, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 2, May 1960, pp. 4-10
AUTHOR'S SUMMARY

Some commentators doubt that Joseph Aspdin invented portland cement, and certainly his 1824 patent fails to distinguish clearly from the prior art. Rival claims of invention were made by I. C. Johnson, and even Aspdin's son William has sometimes been considered a third possibility. In the present survey, the features that distinguish portland cement from earlier cements are pointed out, and records and events subsequent to 1824 are reviewed. The available evidence is scanty, but a judgment is reached regarding the proper assignment of honors under the circumstances.

A short method for the flame photometric determination of magnesium, manganic, sodium, and potassium oxides in portland cement

C. L. FORD, *ASTM Bulletin*, No. 250, Dec. 1960, pp. 25-29
AUTHOR'S SUMMARY

Existing methods for the flame photometric determination of magnesium, manganese, sodium, and potassium have been modified to permit the use of one set of standard calibration solutions for the flame photometric determination of the amounts of all four elements in portland cement. The development of the method is described. Supporting data showing accuracy and precision are presented. Details of the method are given in the appendix.

Rational concrete mix design

BARRY P. HUGHES, *Proceedings*, Institution of Civil Engineers (London), V. 17 (Session 1960-61), Nov. 1960, pp. 315-332
Reviewed by ARON L. MIRSKY

A rational (i.e., mathematical, involving many simplifying approximations, as customary) method for proportioning concrete mixes to meet the following requirements: (1) specified minimum qualities (strength, etc.) in the hardened paste; (2) satisfactory properties (workability, etc.) in the plastic condition; and (3) economic production. Method is based on two characteristics of the aggregate (grading modulus; equivalent mean diameter) and on the concept of specific volume, which for each consituent equals the ratio of the gross apparent volume of that constituent to the total volume of the fully compacted concrete. Advantages claimed for method are that it gives minimum cement content, maximum concrete impermeability, and freedom from segregation.

Ability of various waterproofing membranes to bridge cracks that might develop in concrete on which they are applied

Bulletin, American Railway Engineering Association, V. 62, No. 559, Sept.-Oct. 1960, pp. 153-167
HIGHWAY RESEARCH ABSTRACTS V. 31, No. 1, Jan. 1961

Report contains a description and the results of an investigation of waterproofing membranes fabricated with different combinations of fabrics and felts with asphalts or tars. There was no previous test for determining the behavior of a membrane when a crack develops in the surface to which it has been applied. It was necessary first to develop a method of testing, and second to design an apparatus to permit testing a specimen under controlled conditions to permit measurements of the effect of the particular variable being studied.

Features of membranes that were studied included the kind and grade of bituminous materials, the kind of fabric employed, the number of piles, the bond or slippage strength and the effect of prevailing temperatures.

The temperature existing in a bituminous membrane was a major factor in its performance; therefore, to obtain information of temperature range in membranes in service, recording thermometers were installed on a bridge deck and maintained for 12 months.

Above-ground asphalts with two- and three-ply membranes of cotton fabric proved more satisfactory in the range of 0 to 70 F. Below-ground and synthetic asphalts, even though they stretched more at higher temperatures, did not prove as satisfactory at the lower temperature range as the above-ground asphalts.

Three- and five-ply membranes with felts and above-ground asphalts performed equally well. In comparison to membranes with fabric, the felts were as good or a little better at low temperatures, but the fabric membranes were much better from 40 F and up.

Emulsified asphalts showed little or no crack coverage ability from 0 to 70 F. Cutbacks were little affected by temperature change but did not have much stretch before failure occurred.

Water-reducing retarders for concrete—Physical tests

WILLIAM E. GRIEB, GEORGE WERNER, and DONALD O. WOOLF, *Public Roads*, V. 31, No. 6, Feb. 1961, pp. 136-152

Reports results of tests of the effect of commercial retarders on the properties of portland cement concrete. Included are time of retardation, reduction of water, durability, and compressive and flexural strengths at various ages. Test methods are described and data presented on 25 different commercial retarders.

Water-reducing retarders for concrete—Chemical and spectral analyses

WOODROW J. HALSTEAD and BERNARD CHAIKEN, *Public Roads*, V. 31, No. 6, Feb. 1961, pp. 126-135

In an attempt to standardize testing methods for retarding admixtures, the Bureau of Public Roads investigated the effects of commercial retarders on certain properties of portland cement concrete. The first phase consisted of chemical, ultraviolet, and spectral analyses to determine the composition of each retarder. Conclusions were that: (1) infrared spectral analysis is the best means available for rapid identification and classification of retarders; (2) for specific construction projects, infrared analysis can be used to determine the uniformity of the admixture; (3) all three types of retarders could be distinguished from each other on the basis of infrared spectra; (4) ultraviolet spectra, although less specific than infrared spectra, can be used to identify lignosulfonate retarders and to obtain quantitative information as to their concentration; and (5) although useful in identifying retarder materials, conventional chemical analyses are time consuming and tedious and in many cases yield doubtful results.

Pulverized fuel ash as a construction material

PETER G. K. KNIGHT, *Proceedings*, Institution of Civil Engineers (London), V. 16 (Session 1959-60), Aug. 1960, pp. 419-432

Reviewed by ARON L. MIRSKY

Survey covers the nature of the material, its beneficial effects when used in concrete, and some of its current uses (in the manufacture of pozzolanic cement, lightweight aggregate, brick and block, and in road construction).

Exponential grading of aggregates

W. M. SLATER, *New Zealand Engineering* (Wellington), V. 15, No. 7, July 15, 1960, pp. 232-240

HIGHWAY RESEARCH ABSTRACTS V. 31, No. 3, Mar. 1961

Describes a method of mass concrete mix proportioning used to make concrete of maximum density at a minimum cost for the construction of a gravity dam of about 333,000 cu yd volume, designed and built by the Ministry of Works, New Zealand. Although the head office of the Ministry of Works keeps projects informed on modern dam concrete techniques and directs the work generally, the staff of the individual projects employ concrete mix proportioning methods best suited to local conditions. The method presented in the paper was developed

primarily as a solution to the particular problem of making economic, dense concrete from lightweight aggregate of volcanic origin and has been used with success on two more recent projects. The performance of the concrete in the field is reviewed in the light of contemporary concrete practice and theory.

Occurrence of sulphates in the geological succession

E. M. Hughes, *The Engineer* (London), V. 210, No. 5465, Oct. 21, 1960, pp. 660-661
Reviewed by Aron L. Mirsky

Discusses effect of sulfates on concrete, factors influencing sulfate attack, forms of deposition of sulfates in the ground, and the general distribution of sulfates in the British Isles.

Nature of organic adhesives and their use in a concrete laboratory

William H. Kuenning, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 3, No. 1, Jan. 1961, pp. 57-69
Author's Summary

Unusual properties are available in new classes of thermosetting resins which can be cured at ordinary temperature. The chemistry of these resins permits extensive tailoring of properties to suit the application. Their use has simplified many laboratory operations and has made possible operations which were formerly impossible. Field applications of these adhesives with concrete are also increasing steadily in number and kind.

Wire mesh reinforcing (in Dutch)

S. H. Goudstikker, *Cement* (Amsterdam), V. 12, No. 8, Apr. 1960, pp. 674-679
Reviewed by John W. T. Van Erp

Applications of wire mesh, methods of detailing, schedules and mat lists are given. Systems of marking, labeling, and identification by colors are described. As rules for the use of wire mesh are not included in the standard building codes for reinforced concrete, a building code for wire mesh in full is reproduced, such as has been adopted as standard by the largest municipalities in Holland.

Pavements

Devices for recording and evaluating pavement roughness

F. N. Hveem, *Bulletin* No. 264, Highway Research Board, Jan. 1960, pp. 1-26

Reviews the history of pavement roughness measuring devices and the development of such measurements for analytical purposes by the California Division of Highways. Describes Californian attempts to correlate the readings of roughometers, profilometers and bumpometers to some standard index. Appendix A includes a report on measuring pavement roughness from profilograms while Appendix B is a copy of California's current specifications.

Inspection of a prestressed concrete highway in Austria

Gestrich-Rabe, *Strasse und Autobahn* (Bielefeld, Germany) V. 10, No. 10, Oct. 1959, pp. 418-420
Reviewed by Paul J. Fluss

Based on an inspection by the German Highway Research Society, committee on concrete pavements the author describes two prestressed concrete test sections, 875 yd (800 m) each, of the Autoban Salzburg-Vienna. Abutments and wedges, spaced at empirically chosen distances, were used for prestressing. One section is 6¼ in. (16 cm) thick, and the wedges, which have a taper of 1 to 4, are spaced at 215 ft (197 m). The corresponding figures for the other section are 7⅞ in. (20 cm), taper 1 to 20 and 142 ft (130 m). In the opinion of the Austrian engineers a taper of 1 to 20 is too little, while 1 to 4 is too steep. Travel distance of the wedges and friction forces are the determining factors and actual experience will show the optimum. An array of measuring instruments was built in. So far the following interesting results are available. In one section the center of the slab showed 427 lb per sq in. (30 kg per sq cm) and the wedge 995 lb per sq in. (70 kg per sq cm) prestressing so that 569 lb per sq m (40 kg per sq cm) were taken up by

friction. The movements of the slab ends were so far up to 0.7875 in. (20 mm) that is a total of 1.575 in. (40 mm) at the wedge, which represents a wedge travel distance of 6.3 in. (160 mm) at a taper of 1 to 4. The concrete pavement was built in two layers and is reinforced with wire mesh. Its maximum compressive strength is 8534 lb per sq in. (600 kg per sq cm) and its flexural strength 995 lb per sq in. (70 kg per sq cm).

Effect of granular and soil-cement subbases on load capacity of concrete pavement slabs

L. D. Childs, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 2, May 1960, pp. 11-20
Author's Summary

A laboratory evaluation of the contribution of subbases to the load-carrying capacity of concrete pavements has been in progress for several years. Concrete slabs were built on three thicknesses of each of four granular materials and on two thicknesses of soil-cement. Static loads were applied to the slabs and the resulting deflections, strains, and reactive pressures were compared with those obtained when a slab was built directly on a clay subgrade.

Detailed reports on dense- and open-graded gravel have been published. Paper compares critical results from these early studies with corresponding data from subsequent tests to show the relative benefits of each subbase material. It was found that soil-cement subbases 4 and 5 in. thick made significant contributions to the load-carrying capacity of pavements but similar benefits from granular subbases demanded uneconomical thicknesses.

Measured strains and deflections compared favorably with results predicted by theory. Subgrade pressures were found to be low under loads at all positions except extreme edges and corners. Stresses due to edge loads were reduced appreciably as the load was moved inward from the slab edge, and the critical stress location for wide slabs appeared to be at the transverse joint.

Road surface improvements

D. R. Sharp, *Engineering* (London), V. 189, No. 4905, Apr. 22, 1960, pp. 544-545
Reviewed by Anon L. Mirsky

Brushing (brooming) of fresh concrete results in a surface with improved skid resistance, because the ridges speed runoff of surface water as well as offer a good skid resistant surface. As the ridges wear off, aggregate is exposed and the resulting surface, depending on the original mix, may also become skid resistant. In addition, while light or neutral colored concrete aids driving under adverse seeing conditions, colored concretes and various textures may be used for special effects.

LTS design of continuously reinforced concrete pavement

Benjamin F. McCullough and William B. Ledbetter, *Proceedings*, ASCE, V. 86, No. HW4, Part 1, Dec. 1960, pp. 1-24
Authors' Summary

With the advent of the continuously reinforced concrete pavement, without joints, the highway engineer has been forced to discard existing design theories and rely largely on experimentation and judgment to attempt to properly design this new type of pavement.

This paper is aimed at presenting a rational and uncomplicated approach to the approximate design of continuously reinforced concrete pavement, considering all of the factors and variables that can be evaluated with the engineering tools available. The two major factors to be considered, internal forces developed from restrained pavement volume changes and external forces developed from the traffic loads, are examined, and methods are given whereby each can be evaluated in arriving at an economically safe design. The Load-Temperature-Shrinkage design approach is predicted on the assumption that the concrete should

be designed to withstand the external forces developed from traffic loads, and the reinforcing steel should be designed to withstand the internal forces developed from restrained pavement volume changes. It is shown that a pavement thickness of 7.0 in., with 0.5 percent interior longitudinal hard grade steel over a subgrade whose modulus of subgrade reaction is 100 psi per in., will be sufficient to carry a 16,000 lb design wheel load. Past experiences are cited to indicate that this design approach is adequate. In addition, the areas for possible modification of the approach by experimentation and research are examined.

Precast Concrete

Joints in precast concrete building frames

E. M. RENSAA, *The Engineering Journal* (Montreal), V. 43, No. 8, Aug. 1960, pp. 64-67
Reviewed by ARON L. MIRSKY

Technical and economic aspects of joints in precast concrete frames (beam-beam, column-footing) are discussed. Methods developed by author since 1948 are described and illustrated. Suitability of certain types of joints for winter construction is noted, as is practicability of combination steel-precast concrete structures.

Glass-fibre-reinforced concrete pipes

The Engineer (London), V. 210, No. 5457, Aug. 26, 1960, p. 361
Reviewed by ARON L. MIRSKY

Unreinforced concrete pipe produced by standard spinning techniques is strengthened by external windings of glass fibers. Pipe and fibers are then coated with polyester resin and heat-treated to set the resin.

Under two-point loading, the pipe remains watertight even when cracks extend the full depth of the concrete; recovery on removal of the load is marked.

The coating also affords protection against corrosion and sulfate attack.

Prestressed Concrete

Concrete airport platform prestressed in two directions (in Dutch)

J. W. KAMERLINGH, *Cement* (Amsterdam), V. 11, No. 6, Dec. 1959, pp. 503-506
Reviewed by JOHN W. T. VAN ERP

Platform at Schiphol Airport is 30 x 100 m without joints designed for single wheel loads of 25 tons on 5 sq ft and a temperature differential of 3 deg per in. Pretensioning of the 5-in. slab by single wires was done against the 4 in. concrete underfloor. This was given a bituminous coating to reduce friction.

Influence of elevated temperatures on structural elements of prestressed concrete (Der Einfluss hoher Temperaturen auf Bauteile aus Spannbeton)

TH. KRISTEN and H. J. WIERIG, *Der Bauingenieur* (Berlin), V. 35, No. 1, Jan. 1960, pp. 6-11 (Addendum: Aug. 1960, pp. 327-328)
Reviewed by ARON L. MIRSKY

General discussion and survey of the fire resistance of prestressed concrete structural members. German tests are cited to indicate that concrete is the major factor, not the mild steel reinforcing or the prestressing tendons, with spalling of the concrete being the major ill. Recommended protection is a lightweight concrete fireproofing encasement or other insulation.

Inspection of prestressed concrete

JACK R. JANNEY and RICHARD C. ELSTNER, Prestressed Concrete Institute, Chicago, 1960, 34 pp., $1

Part 1 deals with properties of prestressing steel, placing of steel, measurement of stress, significance of variation in stress and elongation, detensioning, deflected strands, camber, dimensional tolerances, sequence of stressing, and inspection of finished product.

Part 2 includes discussion on the properties of portland cement concrete, water-cement ratio, admixture, batching, creep and shrinkage, aggregates, tests on concrete, slump tests, batch

tests, mixing and placing, curing, vibration, and the checking of beds and form alignment.

Steel for prestressing: Its mechanical properties, selection and testing (Die Spannstaehle, ihre materialtechnischen Eigenschaften, Auswahl und Pruefung)

M. R. Ros, *Schweizer Archiv* (Zurich), V. 25, No. 9, Sept. 1959, pp. 338-346
Reviewed by HENRI PERRIN

Deals with the following questions: what is the best steel quality for prestressed concrete; how to test its ability; how to eliminate defective material. Emphasis is placed on the many and various requirements steel for prestressing has to satisfy. A stress-deformation diagram is not sufficient, but tenacity, resistance to corrosion, and bending resistance are also to be considered leading to the elimination of tempered steel. A comprehensive bibliography is also given.

Some experiences with prestressed concrete piles

STEPHEN VERRALL GARDNER and DUDLEY HOLT NEW, *Proceedings*, Institution of Civil Engineers (London), V. 18, Session 1960-61, Jan. 1961, pp. 43-66
AUTHORS' SUMMARY

Reasons for the application of the principle of prestressing to concrete piles are given, together with some notes on the first prestressed pile to be driven in Great Britain. The use of prestressed piles for river installations, such as fenders and dolphins, is discussed and details of impact tests which preceded work in this particular field are set out in an appendix.

Some figures are given for two static loading tests taken to failure together with relevant driving records and details of ground conditions.

The development of hollow piles for use in jetty work is described, together with details of various tests carried out.

The most important aspect of the paper deals with the formation of tension cracks in the pile shaft under certain driving conditions and examples within the authors' experience are given from which they have deduced recommendations which will overcome trouble from this phenomenon.

Extracts are made from Building Research Report No. 20, which bear out the authors' findings in the field.

Properties of Concrete

Concrete strength near freezing point

J. M. PLOWMAN, *Engineering* (London), V. 189, No. 4891, Jan. 15, 1960, pp. 78-79
Reviewed by ARON L. MIRSKY

Reports results of tests designed to determine the strength-maturity* relationship of concrete at ages between a few hours and 28 days, to extend downward the range of test results which resulted in "Plowman's Law." It was noted that the low-temperature (36 F) curves show a small variation from the normal temperature (64 F) curves; author suggests this might indicate that "one of the chemical reactions involved in the hardening of concrete is accelerated by a reduction in temperature, but that the commencement of this reaction is delayed until the cessation of a second reaction which is itself retarded by low temperatures."

An interesting by-product of the investigation is a method of molding cubes which permits concretes at ages as low as 1 hr to be unmolded successfully.

Discussion, by T. N. W. Akroyd (Feb. 5, 1960, p. 184) and the author (Feb. 19, pp. 248-249) brings up some interesting points.

Discussion (Feb. 5, 1960, p. 184; Feb. 19, pp. 248-249; Mar. 4, p. 313; Mar. 11, pp. 344-345; Mar. 25, pp. 408-409; Apr. 1, p. 441), covers technical aspects of the subject, including disputation concerning the use of "Law" versus "Hypothesis" in science.

* Maturity $= \Sigma\ T \times t$ where T = time, hr and t = temperature in deg F above a datum (11 F).

Drying shrinkage and creep of expanded shale light-weight aggregate concrete

B. J. F. Patten, *Constructional Review* (Sydney), V. 83, No. 11, Nov. 1960, pp. 29-32
Author's Summary

The drying shrinkage and creep characteristics of lightweight concrete made from expanded shale aggregate from a New South Wales supplier, and of concrete made from natural river sand and gravel are compared.

Widening of expansion cracks in mortar by salt solutions

J. A. Roberts and H. E. Vivian, *Australian Journal of Applied Science* (Melbourne), V. 11, No. 4, Dec. 1960, pp. 481-489
Authors' Summary

A new mechanism, by which preexisting cracks in mortar are enlarged and the specimen expanded by salt solutions, is described. Salts containing di- or trivalent positive ions cause marked expansions while those containing monovalent positive ions much smaller expansions. Consequently the expected advantages of applying salts to concrete should be carefully and critically reconsidered.

Factors controlling the strength and shrinkage of concrete

K. M. Alexander, *Constructional Review* (Sydney), V. 83, No. 11, Nov. 1960, pp. 19-28
Author's Summary

A review is presented of a series of interconnected investigations which show that many of the properties of concrete can be explained by studying, in turn, the way in which each of the main constituents determines the characteristics of the hardened mix. On the question of strength for example, it is possible to start by considering not only how strongly cement sticks to itself, but how effectively it adheres to embedded aggregate pebbles. Progressive reduction in the size of the rock particles present raises the question of mineral fillers and pozzolans, and the mechanism by which these materials affect the strength of concrete. The general principles which link concrete strength with the water, aggregate and powdered mineral contents of the mix also explain many aspects of drying shrinkage. It is thus possible to demonstrate the gradual and continuous change in shrinkage characteristics as the composition of a mix is altered from that of a low water-cement ratio portland cement paste, to that of either a straight or blended portland cement concrete, and to explain the mechanism behind such changes.

Structural Research

Experimental studies of the effects of blasting on structures

A. T. Edwards and T. D. Northwood, *The Engineer* (London), V. 210, No. 5462, Sept. 30, 1960, pp. 538-546
Reviewed by Aron L. Mirsky

Controlled blasting tests were performed on six buildings slated for demolition in connection with the St. Lawrence power project. Damage was correlated with size of charge and distance from structure; displacement, velocity, and acceleration; settlement; and strain measurements. Peak velocity was found to give the best correlation.

Ultimate moments and shears in continuous reinforced concrete beams

George C. Ernst, University of Michigan Press, Ann Arbor, 1960, 46 pp., $4

The ultimate moments and shears developed in continuous reinforced concrete structures by loads producing a condition of structural collapse are dependent on plastic deformations in the concrete and the steel at certain critical sections. The ultimate compressive deformation of concrete is only about one-fifth of that at initial strain-hardening of the steel reinforcement, thus complicating both analysis and experimental confirmation, with the possibility of the concrete crushing before the tensile steel can attain sufficient plastic deformation for the plastic collapse of the structure.

This monograph is divided into two parts with Part A presenting the relationships of purely theoretical character, and all experimental results with

comparisons to theory being presented in Part B. The author concludes with a summary of the results from the entire investigation.

Statistical interpretation of simple or compound bending tests

English translation of *Bulletin D'Information*, No. 14, Comité Européen Du Béton, Paris, Feb. 1959, 53 pp. (Limited supply of single copies available from ACI headquarters.)

This bulletin includes the report of CEB Committee No. 1, Tests, which evaluates results of some 1700 European tests *made by members* including those of Steinmann, Torroja, Granholm, and Chambaud; Appendix 1, report of a joint meeting of the committees of tests and eccentric compression; Appendix 2, influence of experimental dispersions on the accuracy of the ultimate moment, and Appendix 3, rectangular distribution diagram in the interpretation of tables of test results of the Rome session.

General

Transmission coefficient for ground vibrations due to blasting

F. J. Crandell, *Journal*, Boston Society of Civil Engineers, V. 47, No. 2, Apr. 1960, pp. 152-168

Reviewed by Aron L. Mirsky

Author previously defined an energy ratio for evaluating risk of damage from ground vibration due to blasting. The proportionality constant in the expression relating the energy ratio to distance and amount of explosive is the transmission coefficient. Paper discusses the various factors affecting this coefficient.

Building research, international

National Academy of Sciences-National Research Council, Washington, D.C., 1960, 42 pp., $1.50

Proceedings of a program conducted as a part of the 1959 fall conferences of the Building Research Institute, Division of Engineering and Industrial Research. Topics covered include: building research in Japan, winter construction in Canada, building research in South Africa, and reports on some of the subjects covered at the International Building Congress held in Rotterdam in 1959. These topics included: tropical construction, standardization of dimensions on the building site, calculation of construction safety factors, research problems with regard to large concrete elements, and documentation of building science literature.

Concerning the adhesion of decorative finishes to Ytong (Zur Frage der Putzhaftung auf Ytong)

Franz Pilny and Werner Struck, *Der Bauingenieur* (Berlin), V. 34, No. 12, Dec. 1959, pp. 456-461

Reviewed by Aron L. Mirsky

Experimental investigations using prismatic bars (which represent a much more severe condition than the usual wall with but one exposed surface), supplemented by some analytical investigations, indicate that trouble with decorative finishes on Ytong ["Current Reviews," ACI Journal, V. 30, No. 2, Aug. 1958, (*Proceedings* V. 55), p. 302] are in general due to excessive initial drying shrinkage of the Ytong rather than to shrinkage of the plaster finish.

Recent investigations in building and architecture (Nové vyskumy v stavebnictve)

Vydavatel'stvo Slovenskej Akadémie Vied, Bratislava, Czechoslovakia, 1960, 286 pp.

This book is a survey of the research work carried out by the Institute of Building and Architecture of the Slovak Academy of Science, Bratislava, Czechoslovakia, during the first 5 years of its existence. The studies covered a wide range of subject matter including precast concrete construction, pozzolans, admixtures, gas concrete, cellular concrete, barite concrete, mix proportioning, lightweight aggregates and concrete, nondestructive testing, welded reinforcement, prestressed foundation slabs, prestressed shells, slab design, prestressed concrete, skew slabs, and other research projects concerned with concrete materials and design of concrete structures.

JULY 1961

***memo from the* President:**

Appreciation

In his "In Defense of Committees" in the May 1960 issue of *ASTM Bulletin,* Robert Burns says ". . . when we have a problem too complex for individual decision, too important to be exposed to the fallibility of the most erudite expert, too comprehensive to be encompassed by the wisest of men, we turn to the much-abused committee, an organism that must of necessity play for percentages, not for miracles; for accomplishment, not for headlines."

With the publication of the new ACI Directory which lists the ACI working committees, it is appropriate to take this special note of them. Did you know that there are more than 50 of these committees?

As an Institute member, a committee member, or a committee chairman have you ever stopped to think just how important technical committees are to the growth of the Institute as a society and to the growth of the concrete industry as a whole?

It would be difficult to say that any one Institute activity is more important than another but there is no doubt that committee activity is one of the most important. Committees have the primary opportunity to carry ACI services and leadership to a still higher level through recommendations, specifications, and standards that will benefit concrete materials and construction practices.

With this importance and opportunity goes the implied responsibility to maintain a continuing activity—for chairman and committee member alike—pressing on toward completion of each assignment for the better guidance of all technicians and engineers, producers, etc., in that specific area. It means finding time to complete the tasks in addition to the usual work load that a member might have. It means keeping on the subject and not letting committee meetings or correspondence get sidetracked into diverse and unrelated discussion. It requires the individual to have integrity, courage to speak, and a recognition that effective committee action usually requires compromise on the part of individual committee members—the end result being a standard, recommendation, or report that will guide what

NEWSLETTER

CONTENTS

Memo from the President 1
1904 Reservoir Rebuilt into Modern Warehouse.. 4
Chapter News 8
Glen Canyon Dam 14
Errata 23
Honor Roll 26
New Members 28
Looking Ahead 28
Bulletin Board 31
List of Advertisers 32

we do today and even mold the concrete practices of tomorrow.

Accordingly we acknowledge the valued leadership of committee chairmen and members. I am sure ACI membership as a whole joins me in expressing appreciation that each one has accepted his assignment, and is looking forward to a renewed dedication as the committee work program moves forward.

Lewis H. Tuthill
President

TAC members extend vote of appreciation to Mather

The Technical Activities Committee noted with unanimous acclaim the devoted services of Bryant Mather, supervisory civil engineer, Concrete Research, U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, Jackson, Miss., as a member and chairman of the group. His 10 years service on TAC has coincided with major changes in JOURNAL schedules and format, important improvements in convention and regional meeting programs, and extensive development of new technical publications. The Institute is indebted to him for his outstanding contributions to its progress.

Mr. Mather is currently a member of the Institute Board of Direction and a member of Committee 116, Nomenclature; Committee 212, Admixtures; and Committee 612, Recommended Practice for Curing Concrete.

Bates addresses Toronto group

ACI Director A. Allan Bates, vice-president, Portland Cement Association, Chicago, was guest speaker at the Ontario Concrete Association meeting, June 13, in Toronto.

At the request of the Toronto group, Dr. Bates spoke on concrete construction in Russia based on his trip to the Soviet Union in the summer of 1960 as a member of an American delegation on an extensive inspection tour. The group visited numerous research institutions, cement and concrete plants, and construction sites in European Russia, as well as beyond the Ural Mountains.

Photo Courtesy Champaign-Urbana Courier

A dinner meeting sponsored by the Urbana area ACI members presented an opportunity for getting together by (l. to r.) Professor Clyde E. Kesler, chairman TAC; Jacob Whitlock, president, Prestressed Concrete Institute; Lewis H. Tuthill, ACI president; William A. Maples, ACI secretary-treasurer; and Chester P. Siess, ACI director

TAC meets in Urbana

At the invitation of Clyde E. Kesler, professor, Department of Theoretical and Applied Mechanics, the Technical Activities Committee held its spring meeting at the University of Illinois, Urbana. In addition to the regular business meeting, held at the Talbot Laboratories, the committee was given the opportunity of touring the laboratory facilities and of visiting the construction site of the unique Illini Assembly Hall.

On Thursday evening the local area ACI members arranged a dinner meeting with TAC at which ACI President Lewis H. Tuthill and ACI Secretary-Treasurer William A. Maples were featured speakers. The subject of the evening was "What is going on in ACI?" and following the formal presentations the local ACI'ers were given the opportunity of questioning the TAC and staff members present and of expressing their views concerning how ACI service might be expanded.

President Tuthill attends Oklahoma Chapter meeting

ACI President Lewis H. Tuthill, concrete engineer, California State Department of Water Resources, was guest at the Oklahoma chapter meeting on May 9. Following a short business session President Tuthill spoke on concreting practices.

Board Committee on chapter activities

The ACI Board of Direction has appointed a committee to study and assist in the development and organization of Institute chapters.

E. A. Finney, director, Highway Research Laboratory, Michigan State Highway Department, East Lansing, is chairman. Members of the committee are: ACI Director A. Allan Bates, vice-president, Portland Cement Association, Chicago, and Samuel Hobbs, secretary-treasurer of the Southern California ACI Chapter, Los Angeles.

1904 Reservoir Rebuilt into Modern Warehouse

THE PERMANENCY OF GOOD CONCRETE was illustrated recently in the construction of a new distributing center in Lyndhurst, N. J., for Becco Chemical Division of Food Machinery and Chemical Corp.

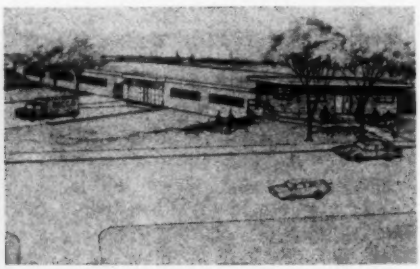
The new Becco facility took advantage of a unique situation to save on the cost of site development and construction of the warehouse. The warehouse occupies part of the former site of the DL&W railroad car and locomotive maintenance shops.

The building was built by roofing over the still standing mass concrete walls of the former DL&W reservoir built in 1904. The reservoir was constructed in the bed of a natural pond and held about 2,000,000 gal of water, which was used to supply the railroad's Kingsland shops for all purposes. Since the railroad ceased using this location in Lyndhurst some 20 years ago the reservoir has stood unused.

The DL&W's engineering department no longer had any construction drawings or details on the concrete mix but records did indicate that a total of 1500 cu yd of concrete were used in the original structure. The concrete walls of the abandoned reservoir averaged 13 ft high. The wall on the north side of the reservoir adjacent to the original pond had a bottom thickness of 6½ ft which tapered to a top thickness of 2 ft. The remaining walls had a top thickness of 1½ ft and a bottom thickness of 4½ ft. The walls were of mass concrete—1904 variety—placed in 40 ft long sections.

For the new warehouse, the walls were increased in height by approximately 4 ft to enable the required amount of window area to be incorporated in the building, which was then roofed over with a flat roof. Openings were breached through the concrete walls with a "skull-cracker" to provide doorways through which trucks can drive right inside the warehouse.

The existing floor of the old reservoir was of solid concrete, ranging from 1 to 4 ft in thickness, providing a sub-



Before (left)—View of the 1904 abandoned reservoir before the walls were increased in height and the new warehouse constructed. After (right)—Artist's sketch of the new Becco distributing center built over a 1904 concrete reservoir

stantial foundation on which several large aluminum storage tanks (for hydrogen perioxide and other peroxygen chemicals) and other facilities were erected within the warehouse.

Van Houten elected ASEE president

Robert W. Van Houten, president, Newark College of Engineering, and director, Newark Technical School, Newark, N. J., has been elected president of the American Society for Engineering Education.

George A. Marston, dean of engineering, University of Massachusetts, Amherst, and Curtis L. Wilson, dean, Missouri School of Mines and Metallurgy, Rolla, were elected ASEE vice-presidents; Wendel W. Burton, Minnesota Mining and Manufacturing Co., was elected treasurer. W. Leighton Collins, University of Illinois, Urbana, continues as secretary.

Officers were installed at ASEE's 69th annual meeting at the University of Kentucky, Lexington, June 26-30.

ICA sponsors 16-week study by highway administrators

A group of 29 administrative highway engineers representing ten different nations, recently began a 16-week study of operations in administrative and laboratory operations in the Ohio Department of Highways under the sponsorship of the International Cooperation Administration, working through the U. S. Bureau of Public Roads.

The engineers are representatives of China, Colombia, India, Indonesia, Japan, Jordan, Korea, Spain, Thailand, and Vietnam; the Ohio Department of Highways is placing these men in various departments. Some of the men will be assigned to work in bridge design, some in hydraulics, some in construction, some in planning, some in traffic, some in maintenance. In these assignments, they will work along with regular engineers of the Ohio Department of Highways.

On their return to their home countries these men will generally move on up into positions of supervisory responsibility, and the lessons they learn here may become the key to their producing better roads in their home countries.

The group begins actual work jobs following a 2-month concentrated orientation course at the Ohio State College of Engineering, in cooperation with Transportation Engineering Center. They will return to their respective countries at the close of their practical experience on September 13.

JOURNAL goes multilingual

Beginning with the July, 1961, issue of the new Proceedings volume, the JOURNAL will carry Spanish, French, and German translations of paper titles and synopses. The synopses will be direct translations of the English synopses, which have long been an important JOURNAL feature, and will appear together following the paper. Papers will continue to be published only in English.

The new step was undertaken to make the JOURNAL of greater value to ACI members overseas.

A large number of ACI members, about one in four, live outside the United States. The additional synopses now make the copies of the JOURNAL already going overseas of greater value to more concrete technologists. Now persons overseas will be able to know as soon as each JOURNAL arrives whether it contains information pertinent to his current undertakings. Spanish-, French-, and German-language publications will also find the synopses of value in keeping their readers informed, through their review and abstract sections, of works published in the JOURNAL.

CONCRETE PERFORMANCE REPORT:

Monolithically-cast POZZOLITH concrete lift slabs speed construction of new "twin" apartment buildings

The new twin 15-story Huron Towers apartments are the tallest to be built in the U.S. with the Youtz-Slick system. The slabs—lifted *two-at-a-time* with 36 hydraulic jacks—are the largest, measuring 215 ft. x 69 ft. x 9 in. and weigh 830 tons each. On the Huron Towers project, each floor was cast *monolithically . . . without joints or "pour strips" of any kind.*

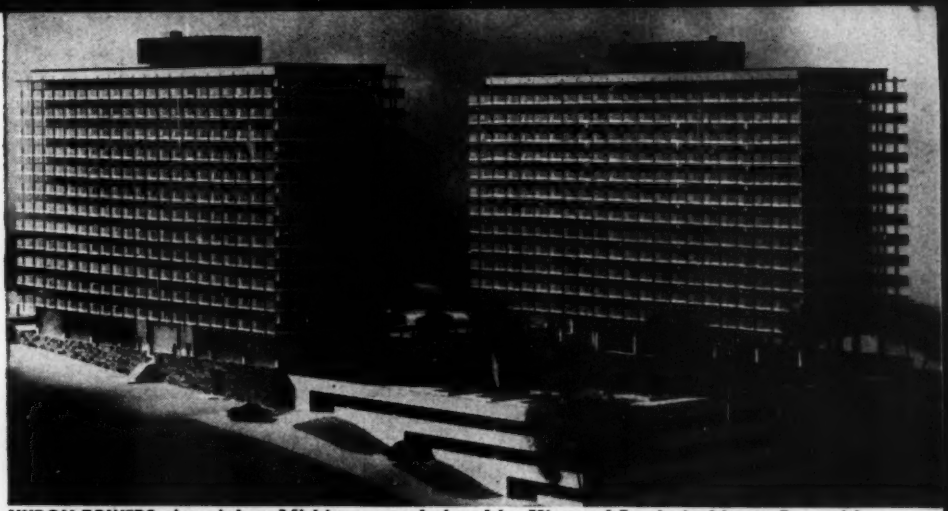
Specifications originally called for a minimum of 6.5 bags of cement for 4000 psi concrete. Preliminary tests, however, showed that the *specified strength could be maintained most economically* by using 5 bags of cement and POZZOLITH . . . and provided the desired reduction in shrinkage that allowed construction of these large slabs without pour strips. This mix design was submitted, approved and used throughout the Huron Towers project. All tests exceeded specified strength.

The contractor developed a unique sequence of casting the 15,000 sq. ft. slabs at floor level, working alternately from one building to the other on a 3-day cycle. While one crew placed reinforcement and conduit in one building, the crew in the other building was placing concrete . . . pumped through an 8-inch line from a centrally-located pump.

LIFT SLAB

POZZOLITH concrete was used throughout both buildings for (1) *ease of pumping* . . . there were no plugged lines during the entire pumping operation (2) *virtually no honeycombing* . . . despite intricate reinforcing steel and electrical and mechanical work in the slabs (3) *minimized shrinkage and cracking* . . . inspection after lifting showed no cracks through the slab.

LOWER BASEMENT FLOORS in both buildings were cast on the ground to serve as casting beds for the liftable slabs. The ground floor and upper basement floor slabs are 14" plus 2½" joist-type slabs spanning onto wide flat beams of the same depth. The 12 apartment floors and the roofs are flat slabs, all 9" thick except the first living floor, which is 10".



HURON TOWERS, Ann Arbor, Michigan, was designed by King and Lewis Architects, Inc., with structural engineering by R. H. McClurg Associates, Inc.—both of Detroit. Project Manager was B. R. Sloan and Project Engineer, Willard C. Reeder—both of Long Construction Company, General Contractor, Kansas City, Mo. Killins Gravel Co. of Ann Arbor supplied the POZZOLITH ready-mix concrete.

TOWERS

MAXIMUM LIFTING HEIGHT of roof slab for this building was 145 ft. For its "twin"—142 ft. The number of liftable slabs—15, in this building, sets a record as the greatest number of concrete decks to be lifted in the United States. Total floor area of the two apartment buildings is 465,000 sq. ft.—431,000 of which were lifted.

Master Builders field men worked closely with the engineer, contractor and ready-mixed concrete producer on the Huron Towers job to aid in achieving the common goal of uniform, high-quality concrete at lowest cost-in-place.

No matter how large, how small, how unique or how conventional the project, call in the local Master Builders field man. He will show you how concrete produced with POZZOLITH is a more durable, versatile building material . . . superior in performance, quality and economy to plain concrete or concrete produced with any other product.

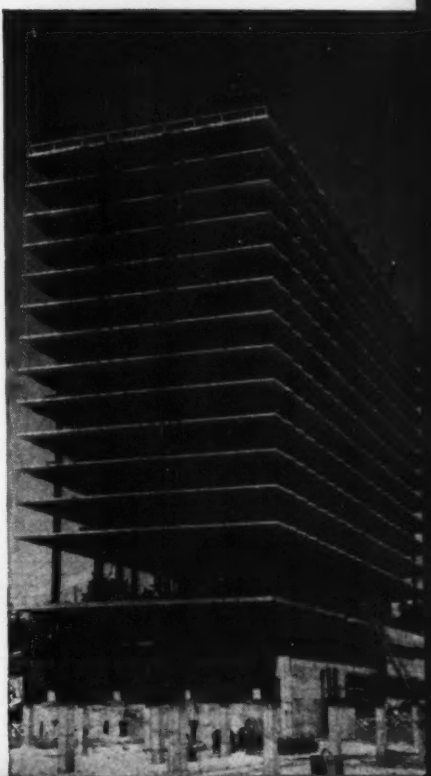
Write for your free copy of the detailed "Huron Towers Concrete Performance Report".

The Master Builders Company
Division of American-Marietta Company
Cleveland 18, Ohio
World-wide manufacturing and service facilities

Our 50th Year

MASTER BUILDERS. POZZOLITH*

*POZZOLITH *is a registered trademark of The Master Builders Co. for its ingredient for concrete which provides maximum water reduction, controls rate of hardening and increases durability.*



CHAPTERS

Southern California Chapter adds ten new members

The Southern California Chapter continues to grow as ten new members were recently added to the chapter membership roster. Chapter memberships were issued to: R. E. Balthaser and P. F. O'Connor, Master Builders Co.; J. N. Sisk and D. C. Troy, Skyline Ready Mix; Ken Herbold, deputy inspector; C. R. Wilder, Portland Cement Association; L. I. Johnstone, Smith-Emery Co.; S. E. Blumenstead, Wailes Precast Concrete Corp.; Louis Fisher, Elastizell, and Albert J. Berger, sales engineer.

An audience of 115 chapter members and guests heard Albert Blaylock, Jr., consulting engineer, San Diego, as guest speaker at the May 18 meeting. Mr. Blaylock spoke on the design and construction of an earth-formed thin-shell reinforced concrete dome of 190 ft span. He performed the structural design and supervised the construction of the dome. Slides were shown to illustrate methods used and unusual features of the project.

The dome has a radius of curvature of about 140 ft and is supported at five points spaced evenly about the circumference with arched openings between the reinforced concrete support blocks. There is a 6 ft circular opening at the crown of the dome. The shell is in general 4 in. thick, with two curtains of reinforcing steel, with thickening of the section at supports and arch openings.

Forming for the dome was accomplished by building a compacted earth mound which was carefully shaped and surfaced with a thin section of lean waste concrete or mortar on which a bond-breaking membrane was applied.

Over this "form" the concrete shell was constructed by placing fine-aggregate lightweight concrete. Mortar pumping equipment was employed in placing the concrete, both in the dome section and in the waste slab.

While larger domes, some earth-formed, have been built, it is believed that this is the longest dome span for such a structure having the limited number and wide spacing of supports which characterizes this project.

Chapter President Byron P. Weintz, chief engineer, Consolidated Rock Products Co., Los Angeles, presided, with Program Chairman Robert E. Tobin, Portland Cement Association, Los Angeles, presenting the technical program.

The next chapter meeting, following the summer season, will be held on

September 21 with Kenneth D. Cummins, ACI technical director, as guest speaker. Mr. Cummins will present an illustrated talk on formwork for concrete.

Northern California Chapter features Lin as guest speaker

T. Y. Lin, chairman, Division of Structural Engineering and Structural Mechanics, and director, Structural Engineering Laboratory, University of California, Berkeley, was guest speaker at the meeting of the Northern California Chapter, May 15.

Professor Lin, author of more than 30 technical papers, spoke on self-stressing cement for precast concrete. Professor Lin is a member of ACI Committee 314, Rigid Frames for Buildings and Bridges; Committee 318, Standard Building Code; and ACI-ASCE Committee 323, Prestressed Reinforced Concrete.

John F. Meehan, Division of Architecture, Sacramento, served as chairman of the technical program.

ACI Michigan Chapter holds meetings in Detroit and Toledo during May

The ACI Michigan Chapter held two meetings in May, one in Detroit and one in Toledo.

Development of a Michigan guide for tolerances of precast concrete was discussed at a luncheon meeting, May 10, in Detroit, with Robert E. Beerbower, division manager, Michigan Flexicore Division, Price Bros. Co., Livonia, Mich., as discussion chairman.

A. M. Haas, professor of concrete technology at the Institute of Technology, Delft, Holland, was the guest speaker at the May 12 meeting at the University of Toledo, Toledo. Dr. Haas compared European and American practices in concrete design and construction.

Dr. Haas has had many years experience in the concrete construction field and has worked in the United States and the Netherlands East Indies, in addition to his native Holland. He has been on the faculty at Delft since 1953. This spring he served as visiting professor in thin shell concrete design at the University of Texas, Austin. A member of ACI, Dr. Haas is also active in several European and international concrete groups.

New ACI Chapters being studied

In recent months various groups around the country have been meeting informally with ACI Secretary-Treasurer William A. Maples and Director A. Allan Bates, vice-president, Portland Cement Association, Chicago, to discuss chapter possibilities.

Informal luncheon meetings have been held in Seattle, Houston, Washington, D.C., New Jersey, and Toronto. Several of the groups plan to hold informal meetings during the coming year and if sufficient interest develops they will petition the ACI Board of Direction for official chapter recognition.

Overseas chapters are also being considered. There has been a recent inquiry from Tokyo about the possibility of organizing a chapter there.

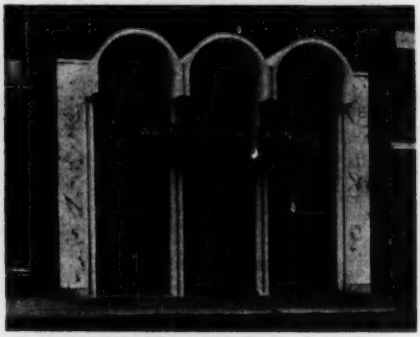
ACI headquarters film for chapter showing

An ACI 20-minute silent, color, 16-mm film with written commentary is available on loan to chapters. The precasting and erection of the folded plate roof is illustrated in sequence of operation.

Please indicate first and second choice for date of showing when requesting film from ACI headquarters.

Steel Scaffolding and Shoring Institute issues safety rules

Recommended Safety Rules have recently been formulated and are being made available by the Steel Scaffolding and Shoring Institute, 2130 Keith Building, Cleveland 15, Ohio. They are designed to promote safety in the use of steel scaffolding. They are intended to deal with some of the many practices and conditions encountered by contractors, subcontractors and industrial plants in the use of steel scaffolding. Twelve of the rules are applicable to fixed scaffolding and cover subjects such as the proper use of sills at the base of the scaffold and the use of guard rails at the top. In addition, the recommendation lists eight safety rules applicable in the erection and use of rolling scaffolds.



Remodeling job uses concrete barrel roof

Here is a remodeling job that used a concrete barrel vault roof—the completely remodeled and reconstructed office of Wayne Federal Savings and Loan Association in Detroit.

First step in the major reconstruction was to tear out most of the interior of the building and reduce it from three floors to one floor. The modern exterior now features three 21-ft high glass panels and topping these is the barrel roof. The three 50-ft long spans are of reinforced thin shell concrete insulated with Styrofoam. It is faced with acoustical plaster and has a white Fibreglas exterior surface.

Karl C. Nelson of Comprehensive Architects and Engineers, Inkster, Mich., directed the design of the project. Hyatt Construction, Inc., Wayne, Mich., was the general contractor.

ACI headquarters construction film

Graduate and senior students at the Michigan College of Mining and Technology, Houghton, recently viewed the ACI film showing the construction of the headquarters building in Detroit, as a part of a lecture series on folded plate design and construction.

This 20-min silent, color, 16-mm film with written commentary is available on loan to chapters and other engineering groups. The precasting and erection of the folded plate roof is illustrated in sequence of operation.

When requesting the film, please indicate first and second choice for date of showing.

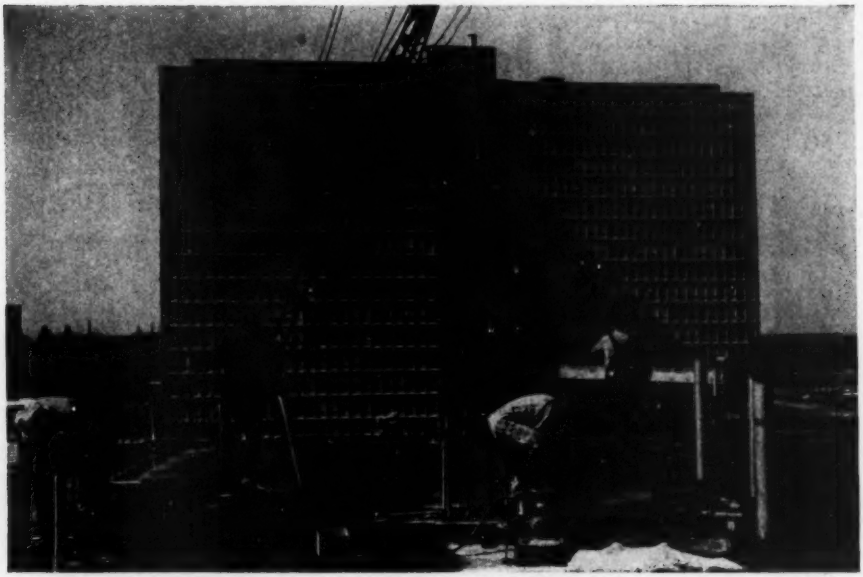
AIA Award

Among the structures receiving a 1961 Award of Merit from the American Institute of Architects was the Willow Creek apartment structure, Palo Alto, Calif. The architects for the apartment were John Carl Warnecke and Associates, San Francisco, and the structural engineer for the project was ACI member Stefan J. Medwadowski, San Francisco. Other awardees were noted in the May 1961 News Letter.

PCA appoints Fields to Vancouver district

Harvey J. Field, Jr. has been named district engineer of the Portland Cement Association's Vancouver, B. C., district office. He succeeds L. T. Willoughby, who has been transferred to the association's general office in Chicago as senior staff assistant.

Mr. Field, a graduate of the University of Kentucky, Lexington, joined PCA in 1952.



Speed - Finish - Strength: All Are Improved With Use of Fly Ash in Concrete

Typical modern apartment buildings are these of the Prairie Shores group now being completed in Chicago. Architect, Loebl, Schlossman & Bennett; engineer, Eugene A. Dubin; contractor, Sumner Sollitt Company.

On this project Fly Ash is incorporated in all concrete mixes, each precise formula designed for its own purpose. Three basic mixes, to test 3,000 psi, 3,750 psi and 5,000 psi—for slabs, beams and columns respectively—account for most of the concrete used.

Fly Ash with its natural "ball bearings" (spherical particles), speeds the concrete into place, helps to fill forms completely. Segregation is reduced to a minimum – especially important where heavy multiple reinforcement is used. Concrete with Fly Ash tends to be homogeneous and to hold together, filling all voids even where percentages of steel are high.

Fly Ash makes for a better finishing concrete. Trowelling is aided by the addition of these "fine-fines" which, because of their spherical shape, give a faster, smoother finish.

The pozzolanic action of Fly Ash in the mix, combining with free lime liberated in the hydration of the cement, and continuing long after the concrete hardens, is a big factor in the excellent final result.

Why not make fly ash a factor in your next job?

Each of the companies below has technical data and competent engineers to help you in designing the most effective mixes employing cement and Fly Ash. We invite your call for more specific information.

WEST PENN POWER COMPANY
Cabin Hill, Greensburg, Pa.

WALTER N. HANDY COMPANY, INC.
P. O. Box 549, Evanston, Ill.

CHICAGO FLY ASH COMPANY
228 N. La Salle St., Chicago 1, Ill.

DETROIT EDISON COMPANY
2000 Second Ave., Detroit 26, Mich.

McNEIL BROTHERS, INC.
P. O. Box 72, Devon Station, Milford, Conn.

Darex Diary

by Mel Prior
Member, American Concrete Institute
Technical Service Manager, Construction Chemicals

GRACE W. R. GRACE & CO./DEWEY AND ALMY CHEMICAL DIVISION
Cambridge 40, Mass. • Chicago 38, Ill. • San Leandro, Calif. • Montreal 32, Canada

(No. 22 of a series)

A Look at Structural Lightweight Concrete

AGGREGATE USED for structural lightweight concrete has been available for over 40 years. Until recently, however, it has not been used extensively by the concrete industry and there is still much to be learned before its maximum potential can be realized.

The nature of this type of aggregate is such that it is virtually unheard of to use it without chemical admixtures. It is inherently angular and porous, requiring a relatively large quantity of paste and/or water for efficient handling and placing.

From the time lightweight aggregates were first introduced for structural purposes, investigations indicated that air entrainment was an absolute necessity for lubricating the mix and maintaining a minimum water/cement ratio. Air entrainment permits the aggregate to move more readily within the mix by minimizing particle interference. However, this is only one phase of the problem since the mix plasticity is also affected by the paste.

More recent research has led to the recommended use of water-reducing admixtures, as well, in lightweight concrete. Such admixtures provide the advantage of plasticizing the paste with a minimum quantity of water. The relatively high bleeding tendency of this type of concrete is thus further reduced.

Since air content is an important factor in lightweight concrete it is necessary to make frequent air determinations. The volumetric method, ASTM Designation C 173-58, sometimes referred to as the Roll-A-Meter method, has proved to be the most satisfactory procedure. The added air is measured without interference from the air which may already be trapped in the pores of the nonsaturated porous aggregate. The need of a correction factor which a pressure type meter would entail is thus eliminated.

Roll-A-Meter for measuring entrained air in lightweight aggregate concrete. Distributed by Charles R. Watts & Co., Seattle, Wash.

On performance, DAREX AEA Air Entraining Agent and WRDA Water-reducing Agent have come to be regarded as essential ingredients in top grade lightweight aggregate concrete.

The combination of these admixtures reduces the water requirement, thereby reducing bleeding and volume change due to drying shrinkage. At the same time, strength is increased with no increase in density. Finishing operations can be started with a minimum delay and yield will be more realistic.

Eney presented 25-year award

William J. Eney, head of the department of civil engineering and the Fritz Laboratory, Lehigh University, Bethlehem, Pa., was presented an award for completing 25 years of service with the university at the Fourteenth Annual Faculty Dinner.

Professor Eney is a member of ACI Committee 115, Research.

Serafim lectures in United States

J. Laginha Serafim, head, Dams Division, Structural Department, Ministerio das Obras Publicas, Laboratorio Nacional de Engenharia Civil, Lisbon, Portugal, has just completed a coast-to-coast lecture tour in the United States.

Here to participate in the Symposium on Basic Research in Civil Engineering Fields Related to the Water Resources, June 12-15, Fort Collins, Colo., he lectured extensively to engineering groups and students throughout the country. During his 18-day stay he visited Columbia University, New York; Massachusetts Institute of Technology, Cambridge; University of Illinois, Urbana, Portland Cement Association Laboratories, Skokie, Ill.; U. S. Bureau of Reclamation, Denver; and Army Corps of Engineers, Washington, D.C.; as well as numerous engineering firms.

Mr. Serafim is a member of ACI Committee 207, Properties of Mass Concrete.

NCMA publication wins award

"Reinforced Concrete Masonry Columns and Pilasters," published by the National Concrete Masonry Association, was selected for honorable mention, Class I, in the First Engineers' Technical and Product Literature Competition sponsored by a joint committee of the Consulting Engineers' Council and the Producers' Council. The award was presented at a special luncheon in conjunction with the annual meeting of the Consulting Engineers' Council, May 4, at the Executive House, Chicago. Robert Dikkers, NCMA design engineer who prepared the manual, represented his association at the ceremony.

Doolittle elected Penn-Dixie president

Fred L. Doolittle has been elected president and chief executive officer of the Penn-Dixie Cement Corp., New York. He succeeds Barton W. Druckenmiller, who has been elected chairman.

Raeder retires

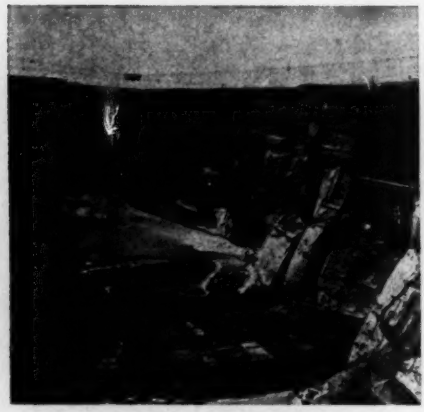
Warren Raeder, emeritus professor of civil engineering at the University of Colorado, Boulder, and visiting professor at the University of Hawaii, Honolulu, for the past year, has announced his retirement from teaching. Professor Raeder, long-time professor and head of the department of civil engineering, University of Colorado, joined ACI in 1929.

Waddell joins Chicago testing laboratory

Joseph J. Waddell has recently joined Soil Testing Services, Inc., Chicago, as chief materials engineer. Until recently, Mr. Waddell was chief materials engineer with Knoerle, Graef, Bender and Associates, Inc., Chicago, consulting engineers for the Illinois Toll Highway Program.

Mr. Waddell is recognized as an expert in concrete materials, including prestressed concrete, having served for 19 years with the Bureau of Reclamation as a concrete and materials control engineer.

He is currently a member of ACI Committee 201, Durability of Concrete in Service; Committee 609, Consolidation of Concrete; Committee 611, Inspection of Concrete; and Committee 612, Recommended Practice for Curing Concrete.



Glen Canyon reaches million

The millionth cubic yard of concrete was placed recently in Glen Canyon Dam, principal storage feature of the Bureau of Reclamation's Colorado River Storage Project. When completed in 1964, the concrete-arch dam will have a total volume of 4,865,000 cu yd and a height of 710 ft, making it the second highest dam in the Western Hemisphere.

Glen Canyon Dam is on the Colorado River, 13 river miles downstream from the Arizona-Utah state line. Construction of the dam and the 900,000-kilowatt powerplant at the toe of the dam began in May 1957 under a $107,955,522 contract awarded to the Merritt-Chapman and Scott Corp., New York.

Early construction activity at the damsite included excavation of two diversion tunnels, construction of cofferdams and diversion of the river, and keyway and foundation excavation and shaping. Construction was halted for 6 months by a labor dispute which began in July, 1959. Work resumed early in January, 1960.

Placement of the first bucket of concrete in the dam was made on June 17, 1960. Concrete placement since that time has been at an average rate of about 40,000 cu yd per week.

Water impounded behind Glen Canyon Dam will form a reservoir, named Lake Powell, which will have a storage capacity of 28,040,000 acre-ft. This will be the largest storage facility in the Upper Colorado River Basin. The water released from the reservoir will meet the obligations of the Upper Basin to the Lower Basin and to Mexico. Sale of electrical energy from the powerplant will help pay for the construction of the 11 participating projects which the Congress authorized with the Colorado River Storage Project in 1956.

Zollman speaks at 65th annual NFPA convention

Charles C. Zollman of Schupack & Zollman, consulting engineers, Newtown Square, Pa., participated in a symposium on Fire Endurance of Prestressed Concrete conducted in conjunction with the 65th NFPA annual convention in Detroit, May 15-19. Phil J. Tatman, Research and Development Laboratories, Portland Cement Association, also delivered a paper at this symposium.

Almost every state in the United States, many Canadian provinces, and a number of foreign countries were represented by the 1500 individuals attending the convention.

Clarkeson elected to AICE membership

The American Institute of Consulting Engineers has announced the election to membership of John Clarkeson, president, Clarkeson Engineering Co., Inc., Boston, and the firm of John Clarkeson, consulting engineer, with offices in Albany, Providence, and New York City.

Mr. Clarkeson, nationally known consulting engineer, has directed the activity of his firm in the planning, design, and supervision of construction of many major highways, municipal improvement, airport, and other projects, as well as assignments in the field of research and development. These

projects have been in the six New England states, Washington, D.C., Virginia, and New York.

Concrete — made entirely from slag?

Concrete made entirely from basic slags from the blast furnace may be a commercial possibility, according to Ontario Research Foundation, Toronto.

Such concrete would use pulverized slag as the cementing material and various sizes of slag as the fine and coarse aggregates. No sand or gravel would be needed.

Exploratory work by J. D. Jones, of Ontario Research Foundation staff, has indicated that fast-setting, high strength concretes can be made by this method. Other advantages are foreseen, including the possibility of lower costs when and if all-slag concretes are accepted commercially.

Blast furnace slags are at present being used as roadbed filler and as aggregates in portland cement concrete. When finely ground, they are sometimes used with portland cement in concrete for the distinct properties they impart to concrete. Pulverization alone is the only treatment required to turn slags into cement.

It is anticipated that all-slag concretes might have special uses in the building industry, and that they could supply a valuable market for an iron and steel industry waste product. More research will be required, however, according to the foundation, to prove the technical and economic feasibility of such a development.

Dauber joins Main staff

Clarence A. Dauber has joined Chas. T. Main, Inc., consulting engineers, with offices in Boston and Charlotte, as head of the firm's thermal power engineering division. Mr. Dauber was formerly director of the civil and mechanical engineering division, The Cleveland Electric Illuminating Co.

"Short" Proceedings volume begins

This July JOURNAL is the first issue of the new Proceedings volume as July issues have been since 1956 when the Institute began publishing 12 JOURNAL issues per year. As a result of action by the Board of Direction in October 1960, however, this July issue represents the last midyear beginning of a volume year. Proceedings V. 58 will be only 6 months long with Proceedings V. 59 beginning in January 1962. Thereafter the Institute Proceedings, represented by the JOURNAL, will be published 12 times per year January through December—with the volume being completed by the following March Part 2 third-quarter discussion and the June Part 2 containing the fourth-quarter discussion, title page, contents, indexes, and errata.

JOURNAL volume number deleted

Another change to be noted beginning with this Proceedings V. 58 is the deletion of the JOURNAL volume number. The JOURNAL will be known only by its Proceedings volume number from now on. The deletion of the dual volume numbers should be welcomed by researchers and librarians who for the past 32 years have found it difficult deciding which number to use. The use of the Proceedings volume numbers preserves the continuity of 58 years of publishing by the Institute.

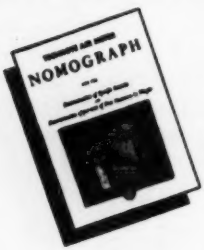
Discussion unchanged

Discussion will still appear quarterly in March, June, September, and December.

TECHKOTE AIR METER..

FIRST CHOICE AROUND THE WORLD

The Techkote Air Meter now provides a multi-range testing instrument that vastly increases the usefulness of the air meter. It is now possible to measure accurately—within a fraction of a degree—entrained air with one instrument on numerous materials such as lightweight concrete, mortar, plaster and soil, in addition to regular concrete.



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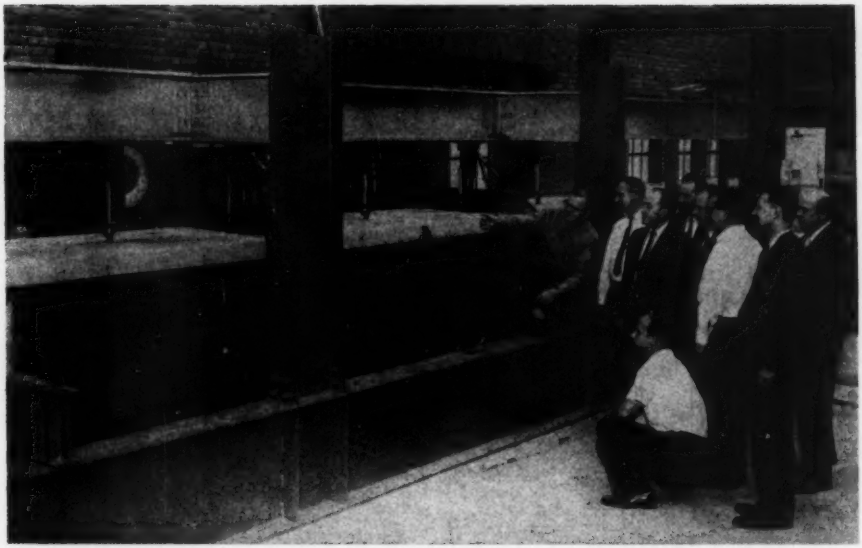


OBSERVING A TEST OF EPOXY RESIN ADHESIVE as a substitute for stud shear connectors in composite construction at the University of Arizona, Tucson, are members of the advisory group to ACI Committee 403, Adhesives for Concrete, which met with part of the committee recently at the university. Pointing out the details of the test on the 21-ft beam with "glued on" slab is Dr. Gene M. Nordby, head of the Civil Engineering Department, director of the university's adhesives research program, and chairman of Committee 403. The observers (standing l. to r.) are J. R. Weschler, Ciba Products Corp.; Walter Hensen, Dow Chemical Co.; R. E. Brown, Shell Chemical Co.; H. C. Klassen, Jones-Dabney Co.; G. M. Scales, Ciba Products Corp.; and Charles Pitt, Union Carbide and Plastics Co., members of the advisory group and B. U. Duvall, U. S. Army Corps of Engineers, member of Committee 403. Kneeling is assistant professor James D. Kriegh, Department of Civil Engineering, University of Arizona, and secretary of Committee 403.

The beam was loaded to failure with hydraulic jacks. It is part of a research program sponsored by the Arizona Highway Department and the U. S. Bureau of Public Roads. Part of ACI Committee 403's contribution to the knowledge of epoxies will be sponsorship of a nationwide, first-hand survey of their use in the United States conducted by Professor Kriegh.

New NCMA research engineer named

The National Concrete Masonry Association, Washington, D. C., has appointed Sidney Freedman as research engineer. In this post, his attention will be primarily directed to research studies currently in progress and will also assist in development and coordination of research projects jointly sponsored by NCMA and other organizations in the construction industry.

Riefenstahl named chairman of PCA safety committee

Howard Riefenstahl, manager of safety and training for Alpha Portland Cement Co., Easton, Pa., has been appointed chairman of the Portland Cement Association's Accident Prevention Committee.

In his post with the PCA committee the Alpha executive will participate in safety projects with 51 committee members representing 178 cement

plants. The committee will conduct regional safety meetings, issue a safety publication, and recommend improved safety training methods.

Mr. Riefenstahl, who has been active in promoting cement plant safety for many years, joined Alpha in 1936. He was named manager of safety in 1946 and manager of safety and training in 1960.

Dutschmann joins Texaco staff

Wilbert C. Dutschmann, a recent graduate of the University of Texas, Austin, has joined Texaco Inc. as an assistant designing engineer at the company's Port Arthur, Tex., refinery.

Are You Up-to-Date on ACI's Bibliography Series!

NO. 1—*Prestressed Concrete* (1955) lists 2100 references to material published from 1896 through 1954. A separate section lists some 60 U.S., British, German, and French patents. Price $2.00. Supplement to 1954 edition, 35¢.

NO. 2—*Evaluation of Strength Tests of Concrete* (1960) annotates 35 selected articles appearing in available technical publications of 1924-1958 dealing with compression tests of concrete, variations in test results, and evaluation of tests. Price $2.00; to ACI members, $1.00.

NO. 3—*Fatigue of Concrete* (1960) describes 114 significant works published since 1898 on the fatigue of plain and reinforced concrete. Price $2.50; to ACI members, $1.25.

AMERICAN CONCRETE INSTITUTE **PUBLICATIONS**

Peltier and Jelley elected to SAME vice-presidencies

Rear Adm. Eugene J. Peltier, chief, Bureau of Yards and Docks, U. S. Navy, Washington, D.C., was recently elected first vice-president of the Society of American Military Engineers. Rear Adm. Joseph F. Jelley, U.S. Navy, Ret., consulting engineer, Colorado Springs, Colo., was elected vice-president for southwestern region.

Col. Walker L. Cisler, president and director of the Detroit Edison Co., Detroit, Mich., heads up the society as president. The officers were installed at the annual SAME meeting on May 15.

Witt addresses ISAS

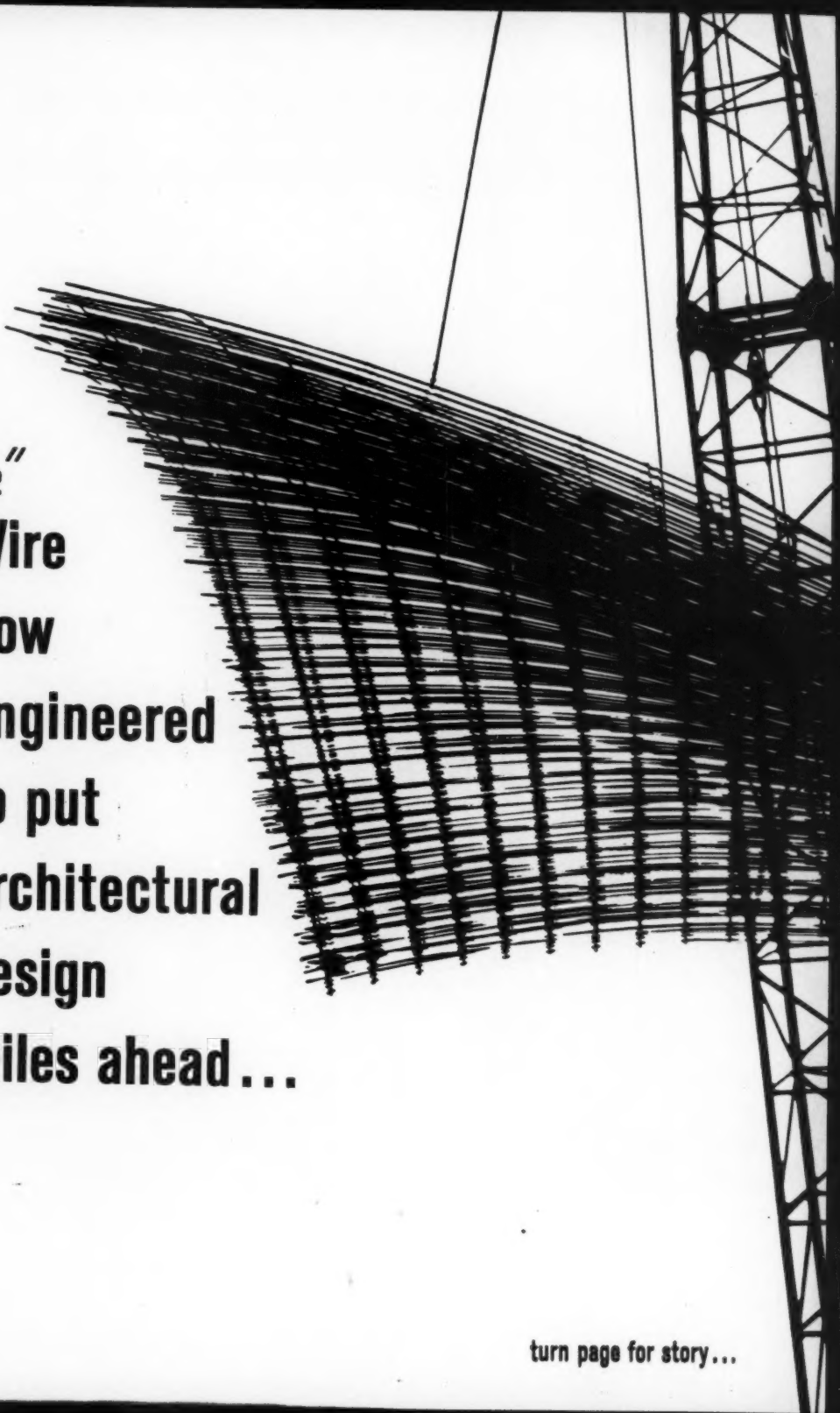
At the 54th annual meeting of the Illinois State Academy of Science, J. C. Witt, consulting engineer, Chicago, presented a paper on research in science and in engineering, and application to industry.

The paper referred primarily to the portland cement industry but was applicable in general to other manufacturing industries, particularly inorganic, nonmetallic industries. Some of the causes of the lag between research and its application, and the confusion that may result from attempted overapplication of research results were mentioned.

Sawyer re-elected head of Lone Star Cement

The directors of the Lone Star Cement Corp., New York, have re-elected H. A. Sawyer chairman of the board and chief executive officer of the company. John H. Mathis, formerly executive vice-president and corporate secretary, was elected president and a director.

Mr. Sawyer served as chairman and president of the firm since January, 1959, and as president since May, 1952. Mr. Mathis has been an officer of Lone Star for 15 years, and has been executive vice-president and secretary since 1958.



½" Wire now engineered to put architectural design miles ahead...

turn page for story...

Now... extra-heavy ½" USS American for design versatility and lower

IN THIN-SHELL HYPERBOLIC PARABOLOID ROOF, SAVES 6 DAYS' CONSTRUCTION TIME

The thin-shell roof of the new library at Hunter College consists of six 60-ft. square inverted concrete umbrellas. They are joined at the edges to form a roof 120 ft. wide by 180 ft. long. Each umbrella is divided into four hyperbolic-paraboloidal quadrants.

Steel reinforcement for the "inside-out" umbrellas was provided by USS American Structural Welded Wire Fabric. Each umbrella used twelve 31' x 10½' Welded Wire Fabric mats.

Installation was easily and speedily made by a small crew. When the concrete work was completed it was found that the use of prefabricated steel fabric had actually saved labor and material . . . and construction time had been cut by six days!

Structural Welded Wire Fabric cost concrete reinforcement

IN THIN FLAT-PLATE FLOOR SLABS, SAVES 15 WORKING DAYS

This handsome 12-story apartment building at 209-223 East 53rd Street in New York City was the first to be constructed with heavy welded wire fabric for reinforcement of thin flat-plate concrete floor slabs. Flat slab floor framing was selected because: (1) the thin (5½") flat-plate slab with its smooth surfaces unbroken by offsets for beams and girders, offers more ceiling height, and (2) it permits flexibility of partitioning and trims plastering and decorating costs.

Structural Welded Wire Fabric was selected to reinforce the slabs because the machine prefabrication of high yield strength steel wires offered: (1) reduction in time and cost of handling 10′ x 20′ prefabricated wire fabric mats as opposed to placing and tying individual reinforcing members—a savings of 1½ days to 2 days per 140′ x 60′ floors . . . and with fewer lathers. (2) Assurance that steel will be placed where required. (3) Positive mechanical anchorage in the concrete to assure crack control. **USS American Structural Welded Wire Fabric is prefabricated with greater accuracy than can normally be relied upon in field work. This assures correct placement and distribution of the steel. Also, the wires are drawn to the very close tolerance of plus or minus 0.003".**

The new high tensile strength (75,000 psi minimum) and high yield strength (60,000 psi minimum) of USS American Structural Welded Wire Fabric permitted a higher working stress for fabric than would have been allowed by the building laws of the City of New York for hot-rolled bars.

USS **American Steel and Wire Division of United States Steel**

see next page

Here's why job-tailored USS American Structural Welded Wire Fabric is your best concrete reinforcement

You can now get USS American Structural Welded Wire Fabric with ½" diameter wires spaced as close as 2" on centers in both directions! These new areas of steel, plus the many time-tested advantages of Welded Wire Fabric, make it the best structural reinforcement for all types of construction.

1. USS American Structural Welded Wire Fabric is made from **cold-drawn** high tensile steel wire. This wire is carefully produced to conform to the requirements of ASTM Specifications A82-58T. The minimum tensile strength is 75,000 psi and the minimum yield, as defined in the specifications, is 80% of the tensile or 60,000 psi. Actually, the cold drawn steel wire has no yield point in the conventional sense of plastic stretch at a constant load. Yield occurs gradually with increasing load beyond the 60,000 psi minimum. This physical advantage of cold-drawn wire makes it **the ideal concrete reinforcement.**

2. USS American Structural Welded Wire Fabric is completely machine prefabricated by electrically welding all intersections to conform to ASTM specifications. This high-strength connection assures positive mechanical anchorage in the concrete.

3. USS American Structural Welded Wire Fabric is prefabricated with greater accuracy than can normally be relied upon in field work. The wires may not vary more than ¼" center-to-center than the specified spacings. This assures correct placement and distribution of the steel. **Wires are drawn to the very close tolerance of plus or minus 0.003".**

4. USS American Structural Welded Wire Fabric requires very little on-the-job tying. Large prefabricated sheets are shipped to the job and placed as a unit. This eliminates thousands of ties and results in important savings.

For more information on the advantages and applications of American Structural Welded Wire Fabric, get in touch with our nearest Sales Office or contact American Steel and Wire, Dept. 1181, 614 Superior Avenue, N.W., Cleveland 13, Ohio.

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United States Steel Export Company, Distributors Abroad

Errata

The following corrections should be made in the discussion of "Resistance to Shear of Reinforced Concrete Beams," which appeared in the June 1961 ACI JOURNAL.

p. 1691—at the bottom of the page change the second last equation "$M = 20000 \times 77 = 1{,}540{,}000$ in.-lb" to "$M = 20{,}000 \times 70 = 1{,}400{,}000$ in.-lb."

p. 1692—in the fourth and fifth line change "$M = 25000 \times 92.7 = 2{,}310{,}000$ in.-lb" to "$M = 25{,}000 \times 91.0 = 2{,}280{,}000$ in.-lb."

p. 1692—in the scond line of the second paragraph, the second line of the third paragraph, and the fourth line of the last paragraph, change "V_u" to "v_u."

p. 1693—in the third line change "f_c/v_1" to "f_c/v."

p. 1693—in the fifth line change "f_c'" to "f_c."

p. 1694—in the fourth and fifth lines from the bottom delete "the two limit cases and are derived from."

p. 1695—in the third line of the second paragraph insert "and V_c" after ". . . T and C'."

p. 1696—in the ninth line change "bending up" to "cutting off."

p. 1696—in lines 13 and 14 delete "or bent up."

p. 1697—in the fourth line of the third paragraph change "Beams 38 and 47" to "Beam 47."

p. 1697—in the third line from the bottom change "chose" to "chosen."

p. 1698—in the fourth line change "appearance of a crack" to "total destruction."

p. 1699—in the seventh and eighth lines from the bottom change "ultimate theory" to "ultimate theory of simple bending."

p. 1701—in the third line of the first paragraph under the heading NUMERICAL EXAMPLE insert "each" after "22 kips."

p. 1702—in the fourth line change "Eq. (14c)" to "Eq. (14d)."

p. 1702—in the seventh line from the top change "920,000/5" to "92,000/5."

p. 1702—in the eleventh line change "K_2" to "k_2."

p. 1703—in the first and second lines change "$Q_u > bd\tau_u$" to "$V > bd\tau_{cu}$."

p. 1704—in the eighth line change "(14c)" to "(14d)."

p. 1704—in the middle of the page change "$K = \frac{vV}{bdf_c}$" to "$k = \frac{vV}{bdf_c}$."

p. 1704—just above Eq. (18) in the middle of the page change "Eq. (14c)" to "Eq. (14d)."

p. 1705—in the fourth line of the fourth paragraph change ". . . distribute the compressive force C' use the shear v or friction factor of concrete." to ". . . distribute the compressive force of C' by v the shear or friction factor of concrete."

p. 1725—in the third line of the second paragraph under "AUTHORS' CLOSURE" change reference 1 to 65.

p. 1725—in the first line of the third paragraph from the bottom change reference 2 to 66.

p. 1726—in the eleventh line of the third paragraph change "the authors did feel that any generally applicable statement could be made" to "the authors did *not* feel that any generally applicable statement could be made."

p. 1727—in the last line of the second paragraph change reference 3 to 67.

p. 1727—in the sixth line of the last paragraph change "it" to "t."

p. 1728—References 61, 62, and 63 should be changed to 65, 66, and 67, respectively.

Paul G. Fluck

Paul G. Fluck, professor in the mechanics department, University of Wisconsin, Madison, recently died at the age of 42.

Professor Fluck had been on the staff at the University of Wisconsin since 1941, except for service in the U. S. Navy and short periods with the Wisconsin Highway Commission Materials Testing Laboratory and the Timber Mechanics Division, U. S. Forest Products Laboratory. He received his PhD in 1949 from the University of Wisconsin.

Professor Fluck, a member of ACI since 1949, coauthored five papers for publication in the ACI JOURNAL. He was a member of ACI Committee 209, Volume Changes and Plastic Flow in Concrete.

He was a registered professional engineer in Wisconsin and a member of Tau Beta Pi, Chi Epsilon, and Sigma Xi.

Eduardo Torroja

ACI Honorary Member Eduardo Torroja, director, Instituto Tecnico de la Construccion y del Cemento (Technical Institute of Construction and Concrete), Madrid, Spain, died June 15.

Professor Torroja received his degree in civil engineering in 1923 and worked in construction until 1927 when he established his own consulting practice. In 1934 he founded the Technical Institute of Construction and Concrete, referred to as one of the world's three outstanding model testing laboratories. The results of tests at the institute formed the basis for most of Professor Torroja's unusual and complicated designs.

Professor Torroja had designed about a thousand projects among which the following are best known: shell roofs for the Algeciras Market Hall, the Recolotes Pelota Hall, and the Zarzuela Horse Racing Track in Madrid; the Esla Viaduct; the aqueducts of Tempul, Alloz, and Tablellina; the bridges of Tordera; the hangars of Torrejon, Barajas, and Cuatro Vientos; the churches of Pont de Suert; and the Fedala elevated water tank.

He had lectured throughout the world on structures and concrete, and had published over 40 papers and books.

Professor Torroja had been a member of ACI since 1946 and was made an honorary member in 1959. He had served as president of the International Federation for Prestressing and as chairman of the International Committee for Shell Structures. He was a member of ASCE, International Council for Building, the Reunion Internationale des Laboratoires d'Essais des Materiaux et de Constructions, European Concrete Committee, the International Association for Bridges and Structures, and many technical societies in Spain.

First meeting of ISO/TC 98 to be held in Warsaw

The first meeting of the International Organization for Standardization Technical Committee 98, Methods of Static Calculations of Building Constructions, will be held at the Palace of Culture and Science, Warsaw, Poland, September 26-29.

Giles elected president of American Cement

James P. Giles has been elected president and chief executive officer of American Cement Corp., Philadelphia, succeeding Walter C. Russell.

Mr. Russell, who has served American Cement and its predecessor companies since 1933, will continue to serve the company as vice-chairman of the board of directors, a member of the executive committee, and as a consultant.

Mr. Giles, who became executive vice-president last year, began his career in the cement industry in 1951 when he joined Hercules Cement Co. as assistant to the president, becoming president in 1958. In that year, Hercules, Peerless Cement and Riverside

Cement merged to form American Cement Corp.

Gillan joins Harrisburg firm

G. K. Gillan, formerly with the Department of Civil Engineering, Pennsylvania State University, is now associated with Modjeski and Masters, consulting engineers, Harrisburg, Pa.

Simpson joins Alpha staff

Jay F. Simpson has joined Alpha Portland Cement Co., Easton, Pa., as director of sales. Prior to joining Alpha, he was director of sales for Barrett Division, Allied Chemical Corp.

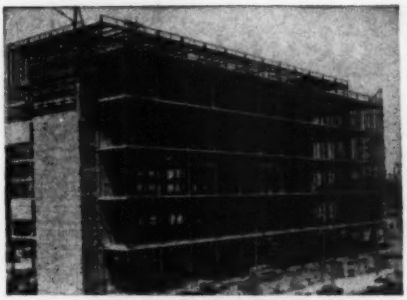
Alpha Cement promotes Tomlinson

Robert W. Tomlinson has been promoted to manager of purchases of the Alpha Portland Cement Co., Easton, Pa. He succeeds the late Ray L. Hamilton.

Shideler speaks at CIB meeting

J. J. Shideler, manager, Products and Applications Section, Portland Cement Association, Skokie, Ill., was guest speaker at the May meeting of the Concrete Industry Board, New York. He spoke on lightweight structural concrete, particularly in the New York City area.

Mr. Shideler is exceptionally active in ACI committee work. He is a member of the Technical Activities Committee; chairman of Committee 717, Practice in Low-Pressure Steam Curing; and a member of Committee 612, Recommended Practice for Curing Concrete; Committee 613, Recommended Practice for Proportioning Concrete Mixes; Committee 623, Cellular Concretes; and Committee 805, Application of Mortar by Pneumatic Pressure.



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Jan. 1—May 31, 1961

In the past 5 months nearly 200 members have added their names to the Honor Roll by proposing new members for the Institute. Have you signed up a new member recently? Among your colleagues working in concrete, there are probably several excellent ACI prospects. Why not tell them about ACI today. Let's keep the membership trend spiraling!

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Order from Publications Department, American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Michigan.

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The Board of Direction approved applications in the following categories: 66 Individual, 3 Corporation, 15 Junior, and 28 Student, making a total of 112 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,375.

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July 3-6, 1961—RILEM Symposium on Durability of Concrete, Prague, Czechoslovakia

July 30-Aug. 13, 1961 — RILEM International Symposium on the Durability of Concrete, Prague, Czechoslovakia

Aug. 11-25, 1961—Prestressed Concrete Conference, Cement and Concrete Association of Australia, Sydney, N. S. W.

Aug. 30-Sept. 2, 1961 — IASS-RILEM Shell Colloquim, Delft, the Netherlands

Sept. 4-6, 1961 — IASS Shell Colloquim, Brussels, Belgium

Oct. 4-5, 1961—Annual Meeting, National Slag Association, Pocono Manor Inn, Pocono Manor, Pa.

Oct. 7-10, 1961—Western Building Industries Exposition, Great Western Exhibit Center, Los Angeles

Oct. 16-20, 1961 — Annual Convention, American Society of Civil Engineers, Hotel Statler, New York

Oct. 15-19, 1961—7th Annual Meeting, Prestressed Concrete Institute, Brown Palace and Cosmopolitan Hotels, Denver, Colo.

Oct. 16-17, 1961—Annual Meeting, The Carolinas Ready Mixed Concrete Association, Inc., Jack Tar Poinsett Hotel, Greenville, S. C.

Nov. 1-3, 1961 — 14th Regional Meeting, American Concrete Institute, Dinkler-Tutwiler Hotel, Birmingham, Ala.

Dec. 5, 1961—First Quality Concrete Conference, University of South Carolina, Columbia, S. C.

Dec. 5-7, 1961—Fall Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

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THIS MONTH

Papers and Reports 1-106

58-1 **Tensile Strength and Diagonal Tension Resistance of Structural Lightweight Concrete** J. W. HANSON 1

58-2 **Prestressed and Precast Concrete Building at Boeing Plant** ARTHUR R. ANDERSON and A. T. WAIDELICH 41

58-3 **Properties of an Expansive Cement for Chemical Prestressing** ALEXANDER KLEIN, TSEVI KARBY, and MILOS POLIVKA 59

58-4 **Design Constants for Interior Cylindrical Concrete Shells** A. L. PARME and H. W. CONNER 83

Concrete Briefs 107-112

Current Reviews 113-128

News Letter 1-32